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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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CONTENTS

Papers: PAGE

The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam, and Hydraulic Tests on Working Models.
By FORD KURTZ, M. AM. SOC. C. E. ... 1953

The Railways' Part in the Development of Hampton Roads.
By CHARLES S. CHURCHILL, M. AM. SOC. C. E. ... 2006

The River and Harbor Problems of the Lower Mississippi: A Symposium:

The Revival of Commercial Transportation on the Mississippi.
By M. J. SANDERS, ESQ. ... 2022

The Economics of Transportation on the Mississippi River.
By JOSEPH E. RANSDELL, ESQ. ... 2028

Discussions:

The River and Harbor Problems of the Lower Mississippi: A Symposium:
By MESSRS. LEWIS MUHLENBERG HAUPT, C. E. GRUNSKY, and WILLIAM T. LYLE ... 2045

The Disintegration of Cement in Sea Water.
By MESSRS. CHARLES A. NEWHALL, EDUARDO DE CASTRO, and R. FERET ... 2060

Improved Type of Multiple-Arch Dams.
By MESSRS. H. J. F. GOURLEY and ALFRED D. FLINN ... 2067

Stresses in Multiple-Arch Dams.
By FRED A. NOETZLI, ASSOC. M. AM. SOC. C. E. ... 2069

Tentative Specifications for Steel Highway Bridge Superstructure: Submitted as a Progress Report by the Special Committee on Specifications for Bridge Design and Construction.
By MESSRS. S. H. LEISTER, THEODORE BELZNER, L. H. SHOEMAKER, and ADOLF EGGENSCHWYLER ... 2072

Reactions for a Particular Type of Unsymmetrical Arch.
By MESSRS. VICTOR H. COCHRANE and J. CHARLES RATHBUN ... 2082

Periodic Fluctuations of Rainfall in Hawaii.
By C. E. GRUNSKY, M. AM. SOC. C. E. ... 2089

Memoirs:

WILLIAM COULSON ARMSTRONG, M. AM. SOC. C. E. ... 2091
EDWARD THOMAS FLAHERTY, M. AM. SOC. C. E. ... 2093
WILLIAM HENRY JAQUES, M. AM. SOC. C. E. ... 2094
ERASMUS DARWIN LEAVITT, M. AM. SOC. C. E. ... 2096
HENRY MANLEY, M. AM. SOC. C. E. ... 2097
HENRY HERMAN MARDEN, JR., M. AM. SOC. C. E. ... 2099
FRED STARK PEARSON, M. AM. SOC. C. E. ... 2100
PRESTON KING YATES, M. AM. SOC. C. E. ... 2102
KINGSLEY PARKHURST ARNOLL, ASSOC. M. AM. SOC. C. E. ... 2103
LOWELL GAYNOR KRIGBAUM, ASSOC. M. AM. SOC. C. E. ... 2104
CLARENCE IVAN LANTZ, ASSOC. M. AM. SOC. C. E. ... 2106
CHARLES WILSON ROSS, ASSOC. AM. SOC. C. E. ... 2107

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

THE HYDRAULIC DESIGN OF THE SHAFT SPILLWAY FOR THE DAVIS BRIDGE DAM, AND HYDRAULIC TESTS ON WORKING MODELS

By Ford Kurtz,* M. Am. Soc. C. E.

To Be Presented March 5, 1924

Synopsis

The object of this paper is to present both the theoretical and the experimental treatments of a new type of shaft spillway for earth dams. Heretofore, shaft spillways seldom have been used and very little information concerning them seems to be available in published form.

The paper treats of the methods of hydraulic design for a large shaft spillway now under construction, the application of these methods to the design of a model of the spillway, and the hydraulic tests on this model, including a discussion of the results of the tests.

On the basis of the data herein submitted, this new type of shaft spillway has been adopted as the most satisfactory and economical construction in the case of a 200-ft. earth dam across a narrow valley, with steeply rising hills on both sides, and it is believed that the data not only justify this particular decision, but also indicate the suitability of the design for any high earth dam similarly located.

General

The Davis Bridge Dam, an earth-fill structure, 200 ft. high, now under construction on the Deerfield River about ¾ mile south of Whitingham, Vt.—a station on the Hoosac Tunnel and Wilmington Railroad—will impound water for a new hydro-electric development of the New England Company's power system and, by using part of the stored waters, will also serve to increase the dry-season flow of the river for other hydro-electric plants of the Company located lower down on the Deerfield River. The valley at the site is so narrow and the sides are so steep that the crest of the dam will be about 50 ft. shorter than the extreme base width of approximately 1 300 ft. (Fig. 1.) The dam will contain about 1 850 000 cu. yd. of earth and will create a reservoir 10 miles long and of a capacity of 5 672 000 000 cu. ft. computed for an elevation of the top of the flash-boards, 6 ft. above spillway level. The dam is being constructed by the so-called "modified hydraulic-fill process". A concrete-lined, diversion tunnel, 1 528 ft. long, with a net cross-sectional area of 396 sq. ft., has been driven through the earth and rock on the east side of the river to take care of the flow of the stream during the construction period. The down-

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **July, 1924**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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stream part of this tunnel is to be utilized for the spillway. These features of the dam and the tunnel are shown on Fig. 1.

The drainage area tributary to the Deerfield River at the dam site is about 184 sq. miles (of which approximately 29.3 sq. miles are also tributary to an existing storage reservoir of the Company on the East Branch of the Deerfield River near Somerset, Vt.). Records of stream flow at several points have been kept by the U. S. Geological Survey for a number of years, and private records have also been kept by the Company at its various plants since their completion. From a study of these data, and after consideration of the effect of the large storage capacity of the reservoir above spillway level in smoothing out the sharp flood peaks, it was decided that the spillway should be designed to have a maximum capacity of 27 000 sec-ft.

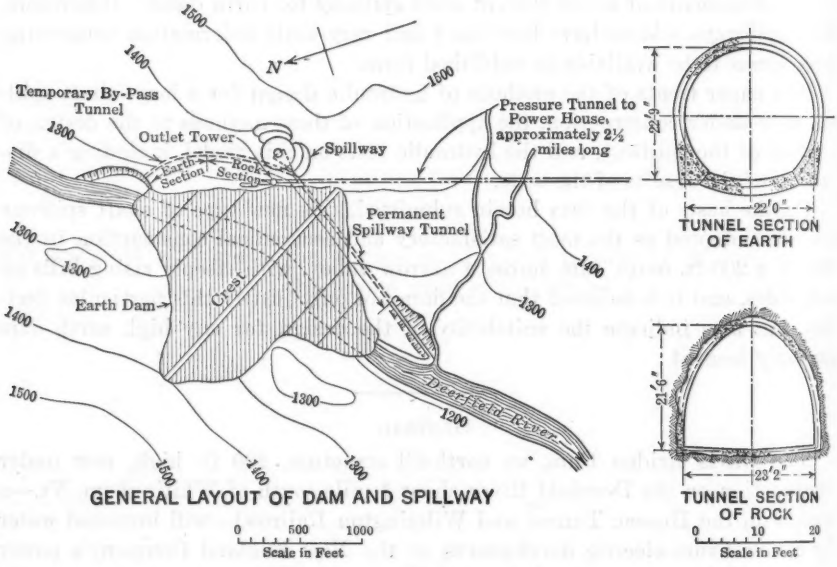

FIG. 1.

At the dam site, the hills on both sides of the river rise to great heights above the crest of the earth dam; at the same time, the lowest point in the range of hills surrounding the reservoir site is more than 100 ft. higher than the crest. The nature of the topography thus prevents the construction of a spillway at a low divide in the surrounding hills—a method which, if feasible, is usually the most economical and satisfactory one for carrying flood waters past an earth dam. Where this method cannot be adopted, it has been customary to construct a masonry overflow spillway at one end of the dam, and the topography usually has been such as to require a spillway channel for conducting the water from the foot of the spillway down the hillside to the bed of the river. In designing and constructing such a channel, every precaution must be taken to protect the earth-fill and the hillside against wash by the flood waters. There are numerous variations of this latter method of design, a common one being a spillway constructed at right angles to, instead of in

line with, the crest of the dam and forming a re-entrant angle in the reservoir. In such a design, the spillway channel also projects into the reservoir, as shown in the construction proposed for the Tieton Dam of the U. S. Reclamation Service.*

A re-entrant spillway having a channel on the hillside was first considered for the Davis Bridge Dam. It was found that the hydraulic problems involved in determining the dimensions of the re-entrant section of the spillway channel could be solved only approximately, requiring, as a result, the adoption of ample dimensions, in order to insure the required capacity. Due to the steepness of the hillsides, these ample dimensions would have involved a large amount of excavation for this re-entrant section of the channel, as well as great expense for the construction of the remaining portion for the drop of nearly 200 ft. As a result, the preliminary estimates of cost for this type of spillway were very high and, therefore, it was decided to investigate the possi-

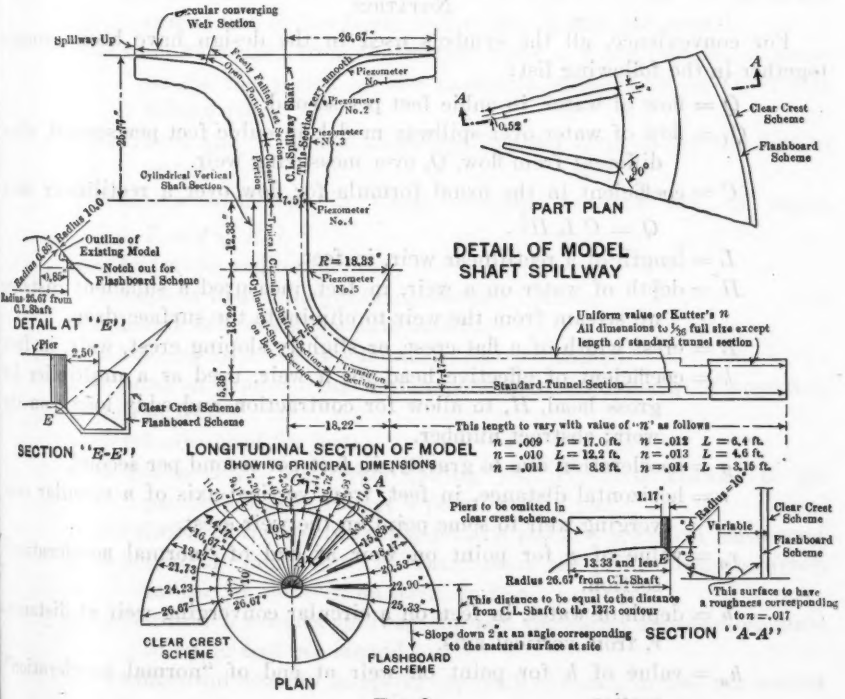
Circular converging Weir Section
Spillway Up
26.67'
Piezometer No.1
Piezometer No.2
Piezometer No.3
Piezometer No.4
C.L. Spillway Shaft
Cylindrical Vertical Shaft Section
Outline of Existing Model
Notch out for Flashboard Scheme
Radius 10.0'
R = 18.33'
Typical Circular Section
Transition Section
Standard Tunnel Section
Uniform value of Kutter's n
All dimensions to 1/50 full size except length of standard tunnel section
This length to vary with value of "n" as follows
n = .009 L = 17.0 ft. n = .012 L = 6.4 ft.
n = .010 L = 12.2 ft. n = .013 L = 4.6 ft.
n = .011 L = 8.8 ft. n = .014 L = 3.15 ft.
DETAIL OF MODEL SHAFT SPILLWAY
PART PLAN
Clear Crest Scheme
Flashboard Scheme
0.52'
30°
DETAIL AT "E"
Pier
2.50'
SECTION "E-E"
LONGITUDINAL SECTION OF MODEL
SHOWING PRINCIPAL DIMENSIONS
PLAN
CLEAR CREST SCHEME
FLASHBOARD SCHEME
Piers to be omitted in clear crest scheme
Radius = 10'
1.17'
Variable
13.33' and less
Radius 26.67' from C.L. Shaft
This distance to be equal to the distance from C.L. Shaft to the 1373 contour
This surface to have a roughness corresponding to n = .017
Slope down 2' at an angle corresponding to the natural surface at site
SECTION "A-A"

FIG. 2.

bility of carrying the flood waters through a vertical shaft into the diversion tunnel previously mentioned, thus eliminating the costly hillside channel. This plan involved the placing of a concrete bulkhead across the tunnel at some point between the foot of the shaft and the up-stream portal after the completion of the dam and when the tunnel is not needed any longer for diversion purposes. Estimates based on a preliminary design of this shaft spillway showed that a material saving would be effected by its use. Consequently, it

* This spillway has been described briefly and illustrated in *Engineering News-Record*, December 1, 1921.

was tentatively adopted, the final decision being contingent on the results of a series of hydraulic tests to be carried out on a working model. The model was built in general to a scale of one-thirty-sixth of the linear dimensions of the full-sized spillway. (Fig. 2.)

On account of the great height of the earth dam and the large quantity of water impounded, it is necessary to have every assurance that the spillway will operate at least as well as the design indicates and especially that it will have the required capacity under the specified head. For these reasons, the satisfactory test of a working model was made a condition precedent to the adoption of the shaft spillway. One objection that has been advanced at times against such a spillway is that it would entrain air in such large quantities as to affect seriously its capacity and general operation. It was desired particularly to investigate this condition by capacity tests, inasmuch as any theoretical investigation would be practically impossible.

Notation

For convenience, all the symbols used in the design have been grouped together in the following list:

Q = flow of water, in cubic feet per second.

Q_1 = flow of water over spillway model, in cubic feet per second, when different from flow, Q, over measuring weir.

C = coefficient in the usual formula for flow over a rectilinear weir, $Q = C\,L\,H^{\frac{3}{2}}$.

L = length of a rectilinear weir, in feet.

H = depth of water on a weir, in feet, measured a sufficient distance up stream from the weir to eliminate the surface drop.

B = crest width of a flat crest, or slightly sloping crest, weir, in feet.

k = coefficient of effective head for a weir, used as a multiplier for gross head, H, to allow for contraction and eddy losses on the weir; abstract number.

g = acceleration due to gravity, in feet per second per second.

r = horizontal distance, in feet, from vertical axis of a circular converging weir to some point on the weir crest.

r_0 = value of r for point on weir at end of "normal acceleration" section.

h = depth of water, in feet, on a circular converging weir at distance, r, from vertical axis.

h_0 = value of h for point on weir at end of "normal acceleration" section.

u = velocity of water, in feet per second, on a circular converging weir at distance, r, from vertical axis.

u_0 = value of u for point on weir at end of "normal acceleration" section.

X = drop in elevation of weir crest between points corresponding to distances, r_0 and r, in feet.

n = Kutter's coefficient of roughness.

h_1 = depth of water, in feet, on a rectilinear flat crest weir after the water surface has become practically horizontal.

z = decrease in depth of water, from value, h_1, at down-stream edge of a flat crest weir, in feet.

x = horizontal distance measured from the down-stream edge of a converging flat crest weir toward the vertical axis of the weir, in feet.

y = vertical distance measured downward from the down-stream edge of a converging flat crest weir, in feet.

a = horizontal distance from the down-stream edge of a converging flat crest weir to the vertex of the parabolic center line of the freely falling jet, measured radially, in feet.

b = vertical distance from the down-stream edge of a converging flat crest weir to the vertex of the parabolic center line of the freely falling jet, in feet.

v_0' = horizontal velocity of the water at the vertex of the parabolic center line of the freely falling jet, in feet per second.

t = time, measured from the instant when a given particle of water passes the vertex of the parabolic center line of the freely falling jet.

c = Hazen and Williams' coefficient in their formula for the flow of water.

c = Chezy coefficient in Chezy formula for the flow of water, $V = c\sqrt{RS}$.

R = hydraulic radius, in feet.

S = slope of hydraulic gradient, abstract number.

l = length of a closed conduit, in feet.

V = velocity in a closed conduit, in feet per second.

D = head on a measuring weir unconnected for velocity of approach, in feet.

h_v = velocity head, in feet.

h' = thickness of the freely falling jet, measured normal to the parabolic center line at a point, x-y, in feet.

v' = velocity of the freely falling jet at a point, x-y, in feet per second.

x_1 = height of "mushroom" of water above the point when the freely falling jet closes in on itself, in feet.

Q_2 = flow which reaches spillway through the spiral approach channel, in cubic feet per second.

V_2 = tangential velocity in spiral approach channel, in feet per second.

Spillway Crests

The first design of the shaft spillway contemplated carrying the maximum flood flow of 27 000 sec-ft., with a depth of water, H, of 7 ft. on the spillway crest which was to be entirely clear, as compared with the final design which has a crest provided with sixteen bridge piers and requires a depth of water on the crest of 8 ft. to discharge the maximum flood flow. The methods used in design were practically the same in both cases. That for the clear crest with a depth of 7 ft. of water will first be described in detail, following which a

brief outline of the modifications necessary to adapt it to the type with flash-board piers, having a depth of 8 ft. on the crest, will be given.

First Design—Flat Crest Type

The section of the diversion tunnel which is to serve as a part of the spillway (Fig. 1) has a "horse-shoe" cross-section, equivalent in area to a circle 22.5 ft. in diameter. It was decided to make the vertical spillway shaft circular in cross-section, to reduce it to a uniform diameter of 22.5 ft. as soon as the water falling over the spillway lip could be accelerated to the required velocity, and to connect this cylindrical shaft to the tunnel by a 90° bend, a part of which would be circular in cross-section and the remainder would consist of transition from circular to "horse-shoe" shape. Experimental data on the excess loss of head due to bends show that of all bends having a radius of less than 10 or 12 diameters the one having a radius of 3.5 to 4 diameters causes the least total excess loss of head. In this case, the practical difficulties in the construction of such a vertical bend compelled the adoption of a radius of about 2.5 diameters, which shape probably causes an excess loss 20% greater than the minimum.

Having fixed the foregoing features of the shaft and its connection to the tunnel, the next step was to design the spillway lip and the converging section of the shaft connecting it with the cylindrical section. The velocity in the latter section at full capacity being 68 ft. per sec., it is evident that the water must have a free, frictionless fall of more than 72 ft., in order to attain it, and the time required to do so would be about 2.12 sec. Inasmuch as the water must have an initial horizontal velocity when it passes the spillway lip, it will take a parabolic course in its free fall. If this free fall should begin at the spillway lip, the initial horizontal velocity with a 7-ft. head could not possibly exceed 17.5 ft. per sec., with the most favorable design of lip, and the discharge per linear foot could not possibly exceed 74 cu. ft. per sec. At this rate for the total discharge, a circular spillway lip with a radius of 58 ft. would be required and the average time for the water to cover the horizontal distance from the lip to the cylindrical shaft would be 3.00 sec. This disagreement between the intervals of time required for horizontal and vertical travel of the water shows: First, that the free fall cannot begin at the spillway lip, inasmuch as the jet would have acquired its final vertical velocity before it had traveled all the way from the lip to the cylindrical shaft; and, second, that because free fall cannot begin at the spillway lip, that lip must be in the nature of a flat crest weir rather than a typical parabolic spillway section of high discharge coefficient.

General Features.—The general type of design decided on for detailed study was made up of the following distinct parts (Fig. 3):

1.—A circular converging weir section with a slightly rounded up-stream edge and having a discharge coefficient equal to that of a rectilinear flat crest weir with a slightly rounded up-stream edge; this equality of coefficients to be obtained by giving to the circular converging weir crest a downward slope just sufficient to com-

pensate for the convergence of the weir, that is, to compensate for the decrease in length of weir as the water approaches the center of the circular weir.

2.—An open portion of a freely falling jet section, beginning at the down-stream edge of the circular converging weir section; the surface to follow the lower nappe of the freely falling converging circular jet and the section to continue downward until the jet "closes in" on itself.

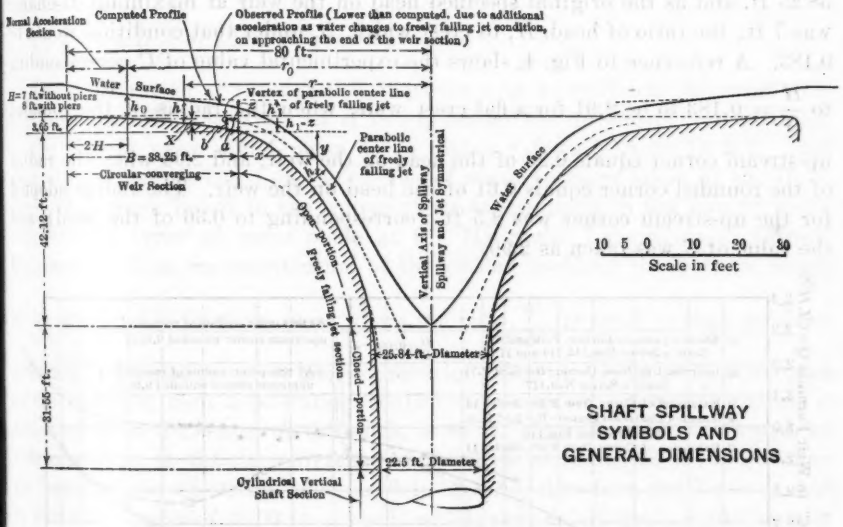
Computed Profile
Observed Profile (Lower than computed, due to additional acceleration as water changes to freely falling jet condition, on approaching the end of the weir section)
80 ft.
Water Surface
Vertex of parabolic center line of freely falling jet
Parabolic center line of freely falling jet
Circular-converging Weir Section
B = 38.25 ft.
Open portion
Freely falling jet section
Closed portion
Vertical Axis of Spillway and Jet Symmetrical
Water Surface
10 5 0 10 20 30
Scale in feet
25.84-ft. Diameter
22.5 ft. Diameter
Cylindrical Vertical Shaft Section
SHAFT SPILLWAY
SYMBOLS AND
GENERAL DIMENSIONS

FIG. 3.

3.—A closed portion of a freely falling jet section, beginning at the lower end of the open portion and continuing downward to the elevation at which the water has acquired a vertical velocity equal to the final velocity required throughout the length of the closed sections of spillway conduit.

4.—A closed cylindrical vertical shaft section.

5.—A closed section of circular cross-section and located on bend.

6.—A closed section on bend, forming a transition from the circular cross-section to the standard tunnel cross-section.

7.—A closed, straight, standard tunnel section, practically horizontal, already designed as a part of the by-pass tunnel for use during the construction of the dam.

Having arrived at these decisions as to general features of the design, the determination of the actual details was a matter of "cut and try", a fact that must be borne in mind throughout the remainder of the description of design methods, to explain how it is possible at times to use certain features and dimensions which are not determined until a later stage of the design. By referring to Fig. 3 which shows the final shape, the various steps in the design may be followed readily.

Design.—In order to have as little loss of head as possible at the spillway lip, a rounded up-stream edge was adopted. The experimental data used for fixing the radius of this rounded edge and for obtaining the proper value for C in the usual weir formula, $Q = C\,L\,H^{\frac{3}{2}}$, were those of Bazin and the U.S. Deep Waterways Board.* The experimental values of C were plotted against corresponding values of the ratio of head on the weir (H) to total width of weir crest (B). The resulting curves are shown on Fig. 4. It will develop later that the width of flat crest adopted for the Davis Bridge Spillway is 38.25 ft. and as the original specified head on the weir at maximum discharge was 7 ft., the ratio of head, H, to crest width, B, under that condition would be 0.183. A reference to Fig. 4, shows the experimental value of C corresponding to $\frac{H}{B} = 0.183$ to be 2.91 for a flat crest weir, when the radius of the rounded up-stream corner equals 0.26 of the head on the weir, and 2.96 when the radius of the rounded corner equals 0.61 of the head on the weir. The radius adopted for the up-stream corner was 2.5 ft. (corresponding to 0.36 of the head), and the value of C was taken as 2.90.

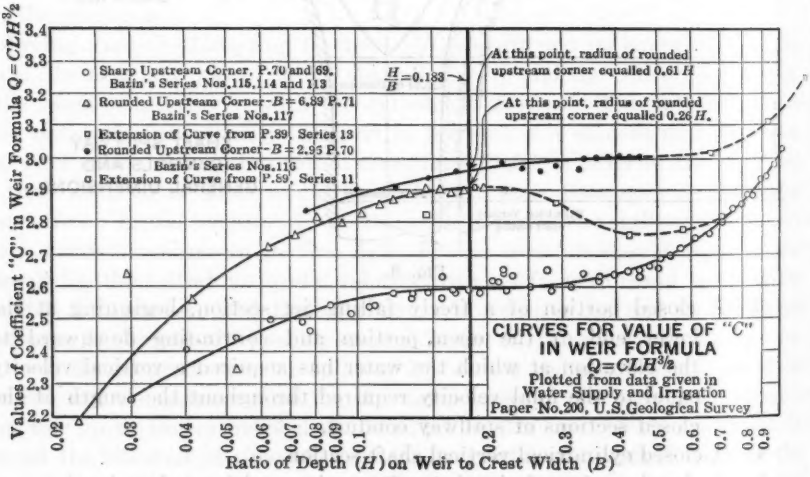
Values of Coefficient "C" in Weir Formula $Q=CLH^{3/2}$
Ratio of Depth (H) on Weir to Crest Width (B)
Sharp Upstream Corner, P.70 and 69, Bazin's Series Nos.115, 114 and 113
Rounded Upstream Corner-$B=6.89$ P.71 Bazin's Series Nos.117
Extension of Curve from P.89, Series 13
Rounded Upstream Corner-$B=2.95$ P.70 Bazin's Series Nos.116
Extension of Curve from P.59, Series 11
$\frac{H}{B}=0.183$
At this point, radius of rounded upstream corner equalled 0.61 H
At this point, radius of rounded upstream corner equalled 0.26 H.
CURVES FOR VALUE OF "C" IN WEIR FORMULA $Q=CLH^{3/2}$
Plotted from data given in Water Supply and Irrigation Paper No.200, U.S.Geological Survey

FIG. 4.

Three of the four variables in the weir formula, $Q = C\,L\,H^{\frac{3}{2}}$, having been fixed, the fourth, which is the length of the clear crest, L, can be found at once to be 502 ft. for $H = 7$ ft. For a circular flat crest weir, the corresponding radius of the up-stream edge is 80 ft.

According to the "principle of least energy", as enunciated by Unwin, and independently also by Frizell, the flow over a flat crest weir, if it takes place without eddies at the up-stream corner and without frictional resistances, follows the law, $Q = 3.087\,L\,H^{\frac{3}{2}}$. If it is assumed that a loss of head equal to $(1-k)\,H$ occurs at the up-stream corner due to eddies, the principle previously mentioned leads to the formula, $Q = 3.087\,L\,(k\,H)^{\frac{3}{2}}$, k being the

* *Water Supply Paper No. 200*, U. S. Geological Survey, pp. 69–71 and 89.

coefficient of effective head. From this formula, it is possible to compute the loss of head at the up-stream corner due to eddies if C is known for any given set of conditions, because $k^{\frac{3}{2}} = \frac{C}{3.087}$. In this case, where C is assumed as 2.90, the value of k is 0.959, that is, the loss of head is 0.041 H, or 0.3 ft., when $H = 7$ ft. Under these conditions, the depth of water on a rectilinear weir of this type after the water surface becomes practically horizontal over the flat crest, would be $\frac{2}{3}\ k\ H$, or 4.48 ft. whereas, the velocity would be $\sqrt{2g\left(\frac{1}{3}\ k\,H\right)}$, or 12.00 ft. per sec.

The acceleration of the water as it passes over a rectilinear flat crest weir continues for a distance from 1.5 H to 2 H beyond the up-stream corner, which, in the present case, would mean for a distance from 10.5 to 14 ft.* This general statement is based on the profiles of water surfaces taken by the U. S. Deep Waterways Board in the course of its experiments on the flow of water over various types of weirs made at the Hydraulic Laboratory of Cornell University. From an examination of the plotted profiles,† the distance seems to be at least 2 H for those cases where the ratio, $\frac{H}{B}$, is small enough to show definitely the distance over which acceleration takes place. If 14 ft. is adopted as the distance, then acceleration continues to a point where the radius, r, measured from the center of the shaft, is 66 ft. At this point, the length of weir crest, that is, the circumference of the circle of radius 66 ft., is less than the length of the up-stream edge, or weir lip, and, therefore, the depth of 4.48 ft. and the velocity of 12.00 ft. per sec., which were determined on the basis of a rectilinear or non-converging crest, cannot obtain under the actual conditions. In order to comply with the condition of a continual decrease in the length of crest as the water approaches the center of the spillway and shaft, and also to maintain the same discharge as would obtain over a rectilinear flat crest weir of length equal to the outer circumference of the circular converging weir, evidently either the depth or the velocity of the water, or both, must increase above the values obtaining under the condition of constant length of crest. The following theoretical treatment of this phase of the problem was used in the design.

If, at the circumference of a circle of radius, r_0, the water has a depth, h_0, and is flowing horizontally (vertical motion so slight as to be disregarded), and radially inward at a velocity, u_0, and if, for a circle of radius, r, the corresponding depth and velocity are designated as h and u, then, for steady flow:

$$Q = 2\,\pi\,r_0\,h_0\,u_0 = 2\,\pi\,r\,h\,u$$

The change in velocity head between these two points equals

$$\frac{1}{2\,g}\left(\frac{Q^2}{4\,\pi^2\,r^2\,h^2} - u_0^{\,2}\right)$$

* *Water Supply Paper No. 200*, U. S. Geological Survey, p. 111.

† *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 329.

If friction is neglected this change in velocity head represents the necessary drop in the elevation of water surface between the two circles of radius, r_0 and r. It follows that the drop in the weir crest between the two circles must be:

$$h + \frac{1}{2g}\left(\frac{Q^2}{4\pi^2 r^2 h^2} - {u_0}^2\right) - h_0 = X$$

The best value of h for economy is that which will require the least drop in the weir crest, that is, which will make X a minimum. This value of h is determined by letting $\frac{dX}{dh} = 0$, or,

$$1 - \frac{Q^2}{4\pi^2 r^2 g h^3} = 0$$

From this equation the following relations are obtained:

$$h = \sqrt[3]{\frac{Q^2}{4\pi^2 r^2 g}}, \quad \text{and,} \quad u = \frac{Q}{2\pi r h} = \sqrt[3]{\frac{Q g}{2\pi r}}$$

also, $\frac{h}{2} = \frac{u^2}{2g}$ = velocity head at any point.

The foregoing treatment shows that both the depth and the velocity vary inversely with the radius for best results, or that both h and u must increase as the water flows toward the center. From the equations, as given, the proper values of h and u for the point at which acceleration corresponding to a rectilinear weir crest has ceased, namely, for $r = 66$ ft., are $h_0 = 5.08$ ft. and $u_0 = 12.76$ ft. per sec. These values are in contrast with $h_0 = 4.48$ ft. and $u_0 = 12.00$ ft., per sec., computed for the rectilinear weir crest. Using a value of $n = 0.014$ in Kutter's formula, and taking the average hydraulic radius as about 6 ft. and the average velocity as about 10 ft. per sec., it can be shown easily that the friction in this 14-ft. section over which "normal acceleration" takes place does not exceed 0 01 ft., which is negligible. Hence, the total drop in water surface from reservoir level to the circle of radius 66 ft., is 0.3 ft. entrance loss plus the velocity head of 2.54 ft. (corresponding to a velocity of 12.76 ft. per sec.), or a total of 2.84 ft. The water being 5.08 ft. deep at this point, the crest of the weir must be 7.92 ft. below reservoir level, or, in other words, the weir crest must drop 0.92 ft. in the 14-ft. width of crest over which "normal acceleration" takes place.

There is no theoretical method of plotting the water surface for the "normal acceleration" section, but its general shape was predicted from the experimental data on profiles of water surface already mentioned. Beyond this section, the complete profiles of both the water surface and the weir crest can be plotted by the use of the formulas for h and u. Just as friction in the "normal acceleration" section was neglected, so the friction for the remainder of the circular converging weir section was found to be so small that it also was neglected.

It is evident that at some particular value of radius, r, the slightly sloping weir crest should end and the inwardly flowing water allowed to fall freely as a converging circular jet of parabolic vertical section. The down-stream face of the weir crest down to its junction with the cylindrical shaft must conform,

of course, to the shape of the lower nappe of this converging jet. The down-stream edge of the weir crest was chosen so that the jet would not close in on itself (at the top of the cylindrical shaft) at such a sharp angle as to produce excessive disturbance due to the loss of its entire horizontal velocity. In order to treat this part of the problem, it was necessary to have a formula for the shape of a jet of water passing over a flat crest weir. Bazin made exhaustive experimental studies of the shape of jets passing various types of weirs, but not of a flat crest weir. The only available data for this type, as already mentioned, are those for the profile of upper nappe only, made by the U. S. Deep Waterways Board. A study of these data, however, was preceded by a theoretical investigation.

If H represents the total head on a rectilinear flat crest weir, h_1, the depth of water on the weir crest after the water surface has become practically horizontal, and $(h_1 - z)$, the depth of water on the down-stream edge of the weir, then, if $(1 - k)\ H$ represents the loss of head at the up-stream edge of the weir,

$$h_1 = \frac{2}{3}(k\,H)$$

$$Q = 3.087\,(k\,H)^{\frac{3}{2}}$$

Velocity of upper filaments of jet as it leaves the down-stream edge of the crest

$$= \sqrt{2\,g\left(\frac{1}{3}\,k\,H + z\right)}$$

Velocity of lower filaments of jet as it leaves the down-stream edge of the crest

$$= \sqrt{2\,g\,(k\,H)}$$

Average velocity of jet (approximate)

$$= \frac{\sqrt{2\,g\,(k\,H)} + \sqrt{2\,g\left(\frac{1}{3}\,k\,H + z\right)}}{2}$$

Depth of jet

$$= h_1 - z = \frac{2}{3}\,(k\,H) - z$$

Then,

$$Q = \left[\frac{\sqrt{2\,g\,(k\,H)} + \sqrt{2\,g\left(\frac{1}{3}\,k\,H + z\right)}}{2}\right]\left[\frac{2}{3}\,(k\,H) - z\right] = 3.087\,(k\,H)^{\frac{3}{2}}$$

$$\left(1 + \sqrt{\frac{1}{3} + \frac{z}{k\,H}}\right)\left(\frac{2}{3} - \frac{z}{k\,H}\right) = 0.7698$$

$$\frac{z}{k\,H} = 0.2267$$

$$\frac{h_1 - z}{k\,H} = 0.44$$

This value of $\frac{h_1 - z}{k\,H}$ agrees very well with experimental data.

Velocity of upper filaments $= 0.75\ \sqrt{2\,g\,(k\,H)}$

Velocity of lower filaments $= \sqrt{2\,g\,(k\,H)}$

Average velocity of filaments $= 0.875\ \sqrt{2\,g\,(k\,H)}$

Average head of filaments $= \frac{\left(0.875\ \sqrt{2\,g\,(k\,H)}\right)^2}{2\,g} = 0.766\ (k\,H)$

Because of the surface curve of the jet of water as it approaches the downstream edge of the flat crest weir, the vertex of that parabola which represents the center line of the jet will not be immediately over the edge of the weir. The unknown distances of the vertex back from and above the edge will be denoted by a and b, respectively, whereas the horizontal distances from the edge to the center line of jet, measured toward the center of the shaft, and the vertical distances measured from the edge downward, will be denoted by $+x$ and $+y$, respectively. If the velocity of the jet at the distance, a, back from the edge is denoted by v_0', the path of a particle of water on the center line of the jet is completely represented by the following equations, in which t represents time measured from the instant when the particle passes the point $(-a, -b)$:

$$x + a = v_0'\,t;\ \text{and,}\ y + b = \tfrac{1}{2}\,g\,t^2$$

Combining these equations,

$$y + b = \frac{(x + a)^2}{\frac{2\,v_0'^2}{g}}$$

Assuming that a and b are small so that v'_0 can be taken as only slightly less than the average velocity of the jet at the edge of the weir and b can be taken as $\left(k\,H - \frac{v_0'^2}{2\,g}\right)$, the equation becomes:

$$y + 0.24\,k\,H = \frac{(x + a)^2}{3.04\,k\,H}$$

When this point had been reached in the theoretical investigation, the experimental data were used to check the correctness of the formula and to obtain the most probable value for a. The value adopted as best fitting the experiments applicable to the present problem was $0.24\,k\,H$, the same as the value of b. The complete formula then becomes:

$$y + 0.24\,k\,H = \frac{(x + 0.24\,k\,H)^2}{3.04\,k\,H}$$

This formula gives $y = -0.22\,k\,H$ for $x = 0$, which is correct, as the thickness of the jet at that point equals $0.44\,k\,H$. Assuming no internal pressure in the jet, the resultant velocity, v', at any point of the center line of the jet is,

$$v' = \sqrt{v_0'^2 + 2\,g\,(y + b)} = \sqrt{2\,g\,(y + k\,H)}$$

This is not exactly true for the portion of the jet up stream from the edge of the weir, as internal pressures do exist there, and, in fact, were assumed in the

derivation of the formulas for h and u for the circular converging weir section. The formula for v', however, is valid for the free portion of the jet, down stream from the edge of the weir.

As $k\,H = \frac{3}{2}\,h_1$ for a rectilinear crest, the equations may also be written as follows:

$$(y + 0.36\,h_1) = \frac{(x + 0.36\,h_1)^2}{4.56\,h_1}$$

and,

$$v = \sqrt{2\,g\,(y + 1.5\,h_1)}$$

For the circular converging weir, h_1 does not have a single value as in the case of the rectilinear flat crest weir, but it was decided that in adapting the foregoing formula to the converging weir, h_1 should be taken as the depth determined by the formula, $\left(h = \sqrt[3]{\frac{Q^2}{4\,\pi^2\,r^2\,g}}\right)$, for the value of r corresponding to the down-stream edge of the slightly sloping weir crest. After a number of trials, the value of r finally adopted for this edge was 41.75 ft. As the radius of the up-stream edge is 80 ft., the width, B, of the flat crest weir, is 38.25 ft., as already noted. The corresponding value of h_1 is 6.90 ft. The equation of the center line of jet is,

$$y + 2.48 = \frac{[\,(41.75 - r) + 2.48]^2}{31.46}$$

The velocity equation is,

$$v' = \sqrt{2\,g\,(y + 10.35)}$$

The down-stream edge of the flat crest weir is 6.90 (depth) + 3.45 (velocity head) + 0.30 (entrance loss) = 10.65 ft. below the water surface of the reservoir.

The velocity, v', and radius, r, of the center line of the jet being known, the thickness of the jet, that is, the depth of water, h', normal to the center line, can at once be found by the formula, $h' = \frac{Q}{2\,\pi\,r\,v'}$. Points plotted on both sides of the center line, and at a distance of $\frac{h'}{2}$ normally from it, determine the surface of the upper and lower nappes of the jet. It should be noted that the jet increases in thickness as it falls, the increased velocity of the water being more than offset by the rapid convergence of the jet. From the plotted curve of the lower nappe of the jet, which fixes the curve of the down-stream face of the flat crest weir, that is, the converging throat section, the radii of the wetted perimeter of the various sections were measured; the hydraulic radii were then computed and friction allowed for on the basis of Kutter's formula with $n = 0.014$. Corrected velocities of jet and corrected depths of jet were then computed for the final determination of the upper and lower nappes.

For the chosen value of H, the upper nappe of the converging jet closes in on itself at a point about 52.77 ft. below the water surface in the reservoir. It

was assumed that, at this point, the entire horizontal velocity of the jet plus a small part of the vertical velocity suddenly is lost by impact and eddies. The residual (vertical) velocity at that point is, therefore, that due to a head of 52.77 ft. less the sum of 0.3 ft., loss of head at entrance, 10.35 ft. of lost velocity head, and 1.04 ft. of friction head in the open portion of the freely falling jet section. The total deductions amount to 11.69 ft., leaving a net head of 41.08 ft., which corresponds to a velocity of 51.4 ft. per sec. The required area of the circular shaft at this level is, therefore, $\frac{27\,000}{51.4}$, or 527 sq. ft., which corresponds to a shaft radius of 12.92 ft. From this point downward, for 31.55 ft., the computations for diameter of shaft, taking friction into account, were made at 10-ft. intervals, the flow being accelerated and the shaft diameter reduced as much as the available head, less friction, would permit. At the lowest point mentioned, which is 84.32 ft. below the reservoir level, the final velocity of 67.90 ft. per sec. is attained, and the top of the cylindrical section of shaft is reached.

The hydraulics of the cylindrical shaft and tunnel are, of course, very simple. There is a straight cylindrical section, 37 ft. long, a curved section, 86 ft. long, including the transition from the circular cross-section to the horseshoe tunnel section, and 832 ft. of straight tunnel section at the end of which is an open channel about 325 ft. long, discharging into the river below the dam site. Assuming that the water level in the open channel is at the crown of the tunnel at the discharge end, the available head required to overcome friction and the excess losses in the bend and in the transition, is about 80 ft., measured from the elevation at which the water attains its final velocity of 67.9 ft. per sec. to the water surface in the open channel. If n in Kutter's formula is taken as 0.014 (equivalent to taking $c = 93.9$ in the Hazen and Williams formula), and if the excess losses in the bend and in the transition are taken as 35% and 5%, respectively, of the velocity head, the total loss of head for a flow of 27 000 sec-ft. is about 73 ft. It was to be expected, therefore, that the tunnel would be capable of discharging more than 27 000 sec-ft. with the head available; tests on the model, described later, bore out this expectation.

In the design of the clear crest type for $H = 7$ ft, no allowance was made for the loss of head in the spiral approach channel, nor for any part of the velocity head corresponding to the spiral or tangential velocity of the water when it reaches the spillway lip and begins its distinctly radial movement toward the center of the shaft. In other words, the first design was made as if the spillway was to be built in the center of a large lake with deep water on all sides, and the setting for the first tests on the working model was an attempt to simulate this condition. The actual location of the spillway on the side-hill is such that the approach to approximately one-half of it is from deep water; the other half, however, is approached by means of a spiral channel. The depth of this channel was fixed at 12 ft. below the spillway crest, which compares favorably with a depth of 13.65 ft. found by adopting the same ratio of depth of channel to head on weir, as obtained in the Bazin experiments,

Series 117, for the condition $\frac{H}{B} = 0.183$, at which time $C = 2.91$. It was realized, of course, that there is a great difference between a straight and a spiral approach channel, but no attempt was made to evaluate this in the first design. The spiral approach channel arbitrarily was assumed to have a tangential velocity of 5.4 ft. per sec., and its dimensions were determined on the basis of this velocity and the depth previously mentioned. The curve joining the spillway lip and the level bottom of the approach channel was adopted on the basis of other published information.* The neglect of these losses in the spiral approach channel undoubtedly accounts in considerable measure for the differences found between the experiments with, and without, such channel. In the design of the pier crest type of spillway for $H = 8$ ft., ample allowance was made for losses, as will be described subsequently, and the predicted capacity of the model was almost in exact agreement with the experimental results. A summary of the computations for the clear crest type of spillway will be found in Table 1.

Final Design—Pier Crest Type

In the final design, full allowance was made not only for friction in the spiral approach channel, but also for losses due to the spiral or tangential velocity of the water. A coefficient of contraction was also applied to the weir constant, C, to take care of the contraction at the sixteen radial bridge piers.

The depth of the channel below the spillway crest was made 12 ft., as in the first design, and the tangential velocity in the channel was assumed to be uniform at 5.4 ft. per sec. In computing frictional losses, the value of n in Kutter's formula was taken as 0.035. It was further assumed that the entire velocity head corresponding to the tangential velocity in the spiral approach channel would be lost at the spillway lip and that, in addition to these two losses, there would be a further loss at the piers due to eddies resulting from the tangential velocity in the spiral. These losses† are summarized as follows:

Entire tangential velocity head lost.	0.45 ft.
Average loss due to friction.....	0.03 "
Head lost at piers due to tangential velocity taken as two-thirds of the tangential velocity head.....	0.30 "
Total	0.78 ft. for half of spillway
Equivalent to.....	0.39 " " entire "

Adopted equivalent net head = 8 — 0.39 = 7.61 ft.

Assuming that there are 16 bridge piers with a clear crest length of 28.25 ft. between piers, and that the end contraction at each pier equals 0.04 H, the end contraction per span amounts to 0.61 ft., which corresponds to a coefficient of end contraction of 0.978. This changes the value of C from 2.91 to 2.845. The entire loss of head due to contraction both on the bottom and at piers

* *Water Supply Paper No. 200*, U. S. Geological Survey, Fig. 3, p. 16.

† For one-half of perimeter only, the other half being open.

TABLE 1.—FIRST DESIGN COMPUTATIONS, CLEAR CREST TYPE.

(Elevation of Water in Reservoir, 1 394.0).

CIRCULAR CONVERGING WEIR SECTION.

r.	h.	u.	h_v.	Elevation of floor, in feet.	Elevation of water surface, in feet.
80	5.55	9.65	1.45	1 387.	1 392.55
66	5.08	12.76	2.54	1 386.08	1 391.16
60	5.41	13.20	2.70	1 385.59	1 391.00
50	6.12	14.05	3.06	1 384.52	1 390.64
41.75	6.90	(14.92)	3.45	1 383.35	1 390.25

FREELY FALLING JET SECTION: (*a*) OPEN PORTION.

x.	y.	Elevation of center line of jet, in feet.	v'.	$h' = \frac{Q}{2\pi x v'}$	Radius of wetted perimeter, in feet.	R, hydraulic radius.	S.	l.	Partial friction loss, in feet.	Accumulated friction loss, in feet.	Corrected v'.	Corrected h'.
Vertex 44.23	—2.48	1 385.83	22.45	....	...	....	.....	...	.....	....	22.45	....
41.75	—2.28	1 385.63	22.75	4.52	42.2	4.47	0.0062	...	.....	....	22.75	4.52
36.75	—0.70	1 384.05	24.90	4.70	37.7	4.57	0.0072	4.7	0.032	0.03	24.86	4.69
31.75	2.46	1 380.89	28.70	4.71	33.0	4.53	0.0097	5.4	0.046	0.08	28.55	4.74
26.75	7.22	1 376.13	33.55	4.79	28.6	4.47	0.0134	6.5	0.075	0.15	33.25	4.82
21.75	13.59	1 369.76	39.20	5.04	23.9	4.58	0.0179	7.8	0.122	0.28	39.00	5.06
19.75	16.59	1 366.76	41.55	5.24	22.1	4.67	0.0198	3.5	0.066	0.34	41.40	5.26
17.75	19.82	1 363.53	44.05	5.49	20.2	4.82	0.0215	3.7	0.077	0.42	43.70	5.53
15 75	23.30	1 360.05	46.50	5.86	18.4	5.01	0.0231	3.9	0.087	0.50	46.15	5.90
13.75	27.06	1 356.29	48.95	6.38	16.6	5.29	0.0241	4.2	0.099	0.60	48.55	6.43
11.75	31.04	1 352.31	51.50	7.10	15.0	5.55	0.0255	4.4	0.109	0.71	51.10	7.14
9.75	35.30	1 348.05	54.20	8.09	13.5	5.86	0.0269	4.7	0.123	0.84	53.60	8.19
7.75	39.82	1 343.53	56.85	9.72	12.3	6.15	0.0280	4.9	0.135	0.97	56.20	9.84
6.75	42.12	1 341.23	58.05	10.98	11.7	6.32	0.0284	2.5	0.070	1.04	57.50	11.05

FREELY FALLING JET SECTION: (*b*) CLOSED PORTION.

Assume that the entire horizontal velocity plus a small portion of the vertical velocity is suddenly lost at Elevation 1 341.23, where the jet closes in on itself. Then, vertical velocity = $8.025\sqrt{42.12 - 1.04} = 51.4$ ft. per sec., and radius of shaft = 12.92 ft. Hydraulic radius = 6.46 ft.; $C = 139$; $S = 0.0212$.

At Elevation 1 332, $v' = 8.025\sqrt{50.31} = 56.9$ (approx.); $R = 12.29$; $S = 0.0272$. Friction loss = 0.22; revised $v' = 56.8$; $R = 12.30$ ft.

At Elevation 1 322, $v' = 8.025\sqrt{60.09 - 0.30} = 61.95$ (approx.); $R = 11.77$ ft.; $S = 0.0344$.
Friction loss = 0.31, which checks the foregoing assumed value.

At Elevation 1 312, $v' = 8.025\sqrt{59.78 + 10 - 0.40} = 66.80$ (approx.); $R = 11.34$ ft.; $S = 0.0420$.
Friction loss = 0.38, which checks the foregoing assumed value.

At Elevation 1 309.68, $v' = 8.025\sqrt{69.38 + 2.32 - 0.10} = 67.90$; $R = 11.25$ ft.; $S = 0.0436$.
Friction loss = 0.10, which checks the foregoing assumed value.

can now be determined from the new value of C, and amounts to 5.4%, or 0.41 ft. The necessary length of spillway, L, to pass 27 000 sec-ft., with the gross head of 8 ft., after deducting all the foregoing losses, is found to be 452 ft., which corresponds to 16 spans of 28.25 ft. each. The thickness adopted for each pier at its outer end being 3 ft., the total required circumference of spillway is 500 ft., corresponding to a radius of 79.4 ft. This is practically the same radius as that adopted for the clear crest spillway in the first design. If the crest were rectilinear, the depth of water after the water surface becomes practically horizontal would be $\frac{2}{3} k H$ (in which, $H = 7.61$ ft. and $k = 0.946$), or 4.8 ft., whereas, the velocity would be $\sqrt{2 g \left(\frac{1}{3} k H\right)}$, or 12.43 ft. per sec.

In adapting to the case of the spillway with bridge piers the formulas for h and u previously used for the circular converging weir section after "normal acceleration" is completed, it is evident that the variable length of crest, $2 \pi r$, previously used, should be replaced by $\frac{28.25}{31.25} (2 \pi r)$, assuming the bridge piers to have radial faces and to extend to the theoretical down-stream edge of the flat crest weir, at which point the free fall of the jet begins. With this single change, all the formulas for the first design become applicable to the final design and from them was computed the profile of the converging throat of the spillway down to a junction with the cylindrical shaft. The value adopted for B, the theoretical width of flat crest, was 42 ft.

The differences between the first and final spillway designs, both as to diameter of spillway lip and shape of the profile, were so slight that the values of the first design were ultimately adopted, except for a slight modification of profile near the spillway lip, in the form of a flat annular space which makes it possible to have a straight line of flash-boards between adjacent piers and, at the same time, have the bottoms of the boards level.

Design of Model

The size of the model was influenced both by the available water supply at places where it was to be tested and by considerations of cost. It was decided to build the model in general to $\frac{1}{36}$ of the linear dimensions of the full-sized spillway, with only such distortions from that scale as might be necessary, in order to maintain a fixed ratio between the flow through the full-sized spillway and the flow through all parts of the model. For this purpose, the following study for the design of the model was made.

1.—*Flow over Flat Crest Weir.*—If the spillway lip is an exact linear model of the full-sized spillway, the ratio, $\frac{H}{B}$, remains constant if the experimental heads are $\frac{1}{36}$ of the actual heads, and, hence, values of C can be assumed to remain constant. Therefore, the flow ratio will depend on the respective values

of $L H^{\frac{3}{2}}$ and will equal $\frac{1}{36}\left(\frac{1}{36}\right)^{\frac{3}{2}}$, or $\frac{1}{7\,776}$. Velocities will obviously vary with $H^{\frac{1}{2}}$, that is, the velocity ratio will be $\frac{1}{6}$.

2.—Flow over Circular Converging Weir Section.—The formulas already derived for the relation between Q, h, and u for this section show that Q varies as $(h^3 r^2)^{\frac{1}{2}}$ and v varies as $\frac{Q}{hr}$. If the flow ratio is $\frac{1}{7\,776}$, and the dimension, r, follows the model ratio, $\frac{1}{36}$, the depth ratio will be $\frac{1}{36}$, the same as the model ratio, and the velocity ratio will be $\frac{1}{6}$. It is seen, therefore, that there need be no dimensional distortion in this section.

The matter of friction ratio for this section still remains to be investigated. Assuming for the full-sized spillway a coefficient of roughness, n, in Kutter's formula of 0.014 and an average hydraulic radius of about 4 ft. for the maximum flow of 27 000 sec-ft., the corresponding value of c in the Chezy formula is 132. The slope of the energy gradient for this section of spillway varies as $\frac{u^2}{c^2 R}$ and should be the same in the model as in the full-sized spillway, as this slope is a ratio or abstract number. Inasmuch as the ratio, $\frac{u^2}{R}$, remains constant as has been noted, it is evident that the energy gradient can remain constant only if c is the same in both the model and in the full-sized spillway. As c depends on the hydraulic radius (the slope of energy gradient remaining constant), and as the hydraulic radius for the model is only $\frac{1}{36}$ of that for the full-sized spillway, it is obvious that c can only be made the same for both, by changing or "distorting" the value of n in Kutter's formula for the model and that even then c can be made the same only for some particular discharge through the spillway. If the chosen discharge is the maximum flow of 27 000 sec-ft., the hydraulic radius for the model is 0.11 ft. and, for $n = 0.009$, the value of c is 112. This is the lowest value of n given by Kutter so that, in order even to approach frictional similarity, it was necessary to make this part of the model as smooth as practicable. Fortunately, the influence of friction on the design is so slight that the impossibility of simulating conditions, as between the model and the full-sized spillway, is of minor moment.

3.—Freely Falling Jet Section.—The formula for velocity in this section shows that it varies as $\sqrt{y + kH}$. It has already been seen that H for the model is $\frac{1}{36}$ of the head for the full-sized spillway; y is, of course, a linear dimension of the spillway, for which the ratio is also $\frac{1}{36}$. The velocity ratio for this section is, therefore, $\frac{1}{6}$. If the flow ratio is $\frac{1}{7\,776}$, it follows from the for-

mula, $\left(h' = \frac{Q}{2\,\pi\,r\,v'}\right)$, that the depth ratio will be $\frac{1}{36}$, the same as the model ratio. Therefore, no dimensional distortion is necessary for this section.

As far as the friction ratio is concerned, it was found that, for the maximum flow of 27 000 sec-ft., the hydraulic radius of the full-sized spillway varies from 4.47 to 6.32 ft., and that, for a value of $n = 0.014$ in Kutter's formula, the value of c in the Chezy formula is 137. The hydraulic radius for the model varies from 0.124 to 0.176 ft. and with $n = 0.009$, the average value of c is 121. Here, again, it was necessary to make the model as smooth as possible, although the effect of friction is a minor factor.

During the course of the tests, certain phenomena appeared, which it was desirable to interpret as to their probable physical magnitudes in the full-sized spillway. These were (*a*), the "mushroom" of violently agitated water formed in the cone-shaped space above the freely falling, impinging jet (Figs. 7 and 9); (*b*), the projecting jet of water formed opposite the "nose" of the spiral approach channel in the case of the model of the clear crest type, due to the spiral movement of the water (Figs. 9 and 19); and (*c*) the partial vacuum in the lower portion of the freely falling jet. The following approximate interpretations have been made.

(a).—Mushroom of Disturbed Water.—As all flows in the model are $\left(\frac{1}{36}\right)^{\frac{5}{2}}$ times those in the full-sized spillway and all velocities are $\left(\frac{1}{36}\right)^{\frac{1}{2}}$ times full-sized velocities, and as the power loss due to shock or impact must be proportional to $Q\,(v')^2$, it follows that the power loss ratio is $\left(\frac{1}{36}\right)^{\frac{7}{2}}$. If this lost power is expended in maintaining in motion a "mushroom" of water of height, x_1, above the point where the jet closes in on itself, the volume of this mushroom varies as ${x_1}^3$, whereas the average height of throw varies as x_1. The power expended in maintaining the mushroom, therefore, varies as ${x_1}^4$, from which it is to be expected that the height ratio of the "mushroom" would be $\left(\frac{1}{36}\right)^{\frac{7}{8}}$, or about $\frac{1}{23}$; in other words, at corresponding stages of flow, the height of the mushroom in the full-sized spillway will not be thirty-six times, but about twenty-three times, the height on the model.

(b).—Projecting Jet, Clear Crest Model.—The lost power used in projecting this jet horizontally is undoubtedly proportional to $Q_2\,{v_2}^2$, where Q_2 is the flow which reaches the spillway through the spiral approach channel and v_2 is the tangential velocity in the channel. Evidently, the power loss ratio is $\left(\frac{1}{36}\right)^{\frac{7}{2}}$, as in the case of the "mushroom", but, in this case, the power is used rather to increase by an unknown amount the horizontal velocity of an unknown volume of the flow near the nose. There seems to be no way of determining just how these two unknown factors will arrange themselves, in order to

account for the lost power. If the flow ratio for this projected jet is $\frac{1}{7\,776}$, and the velocity ratio is $\frac{1}{6}$, the "power expended" ratio will be the same as the "power lost" ratio, and the dimensional ratio for the projected jets will be $\frac{1}{36}$, the same as the dimensional ratio for the model itself, but this is only one of an infinite number of possible sets of ratios.

(c).—Partial Vacuum.—As explained in the case of the first design of the spillway, the pressure within the freely falling jet section down as far as the top of the vertical cylindrical section should be atmospheric. In order to check this, at first a single piezometer was connected to the model at the top of the cylindrical shaft. When the first day's experiments at Worcester indicated that, for flows somewhat in excess of the design maximum, a considerable vacuum existed, it was decided to study further the pressure conditions within the model; therefore, four additional piezometers were installed at points shown in Fig. 2. Succeeding observations plainly indicated that, as far as the model was concerned, the stream of water, after the jet closed in, accelerated more than had been anticipated so that the jet was smaller than the shaft of the model. For some unknown reason, there was a natural Venturi tube effect, and the velocity head in excess of that assumed in the design must be regained as pressure head beyond the last piezometer, for if it was lost in shock or eddies there would not be sufficient pressure head remaining to account for the observed flow. Although all these conclusions were obvious in the case of the model, the very fact that no reason could be discovered for this Venturi effect precludes any interpretation of the model results as far as the full-sized spillway is concerned. It is believed that there will be a partial vacuum in the full-sized spillway when the flow is equal to or approaching the maximum capacity, but the extent of the vacuum cannot be predicted. If the dimensional ratio of the Venturi jets should be the same as that between the model and the full-sized spillway, it can be shown that the vacuum ratio has the same value. However, this could only be the case for measured vacuums of 0.944 ft., or less, on the model for, obviously, a vacuum in excess of 34 ft. cannot be obtained in the full-sized spillway.

4.—Cylindrical Shaft, Bend, Transition, and Standard Tunnel Section.—If this part of the model is built to $\frac{1}{36}$ of the linear dimensions of the full-sized spillway with the sole exception of the length of the standard tunnel section, it will be shown that the flow ratio of $\frac{1}{7\,776}$ can be maintained. It is evident that the available head in this section of the model is $\frac{1}{36}$ of the head on the full-sized spillway. Furthermore, as excess loss of head in bends and transition is assumed to vary with the velocity head, $\frac{V^2}{2g}$, it is seen that if the velocity in the bend and transition is $\frac{1}{6}$ of that in the full-sized spillway, the ratio of excess

losses will be $\frac{1}{36}$, the same as the model ratio. The remaining problem, therefore, is to make the total friction loss ratio also $\frac{1}{36}$.

If the Chezy formula for friction loss is used, the total loss in this section due to friction varies as $\frac{V^2 l}{c^2 R}$, in which l = the total length of the section and R = the hydraulic radius. In order to obtain a total friction loss ratio of $\frac{1}{36}$, with a velocity ratio of $\frac{1}{6}$ and a hydraulic radius ratio of $\frac{1}{36}$, then $\frac{l}{c^2}$ for the model must be $\frac{1}{36}$ of $\frac{l}{c^2}$ for the full-sized spillway. The value of R for the latter is 5.3 ft. which, with $n = 0.014$ in Kutter's formula, corresponds to $c = 136.5$. The value of l for the full-sized spillway being 957 ft., it follows that $\frac{l}{c^2}$ for the model must equal $\frac{1}{36} \times \frac{957}{136.5^2}$. The hydraulic radius for the model being 0.147 ft., the values for n and l given in Table 2, can be obtained for the model.

TABLE 2.

Kutter's n.	Chezy's c.	Total length of section, l, in feet.	Length of standard tunnel section, in feet.
0.009	120	20.4	17.0
0.010	105	15.6	12.2
0.011	93	12.2	8.8
0.012	83	9.8	6.4
0.013	75	8.0	4.6
0.014	68	6.55	3.15

If not distorted, l would be 26.3 ft. Inasmuch as it was desired to have the minimum amount of distortion, it was decided to make this whole section as smooth as possible (for $n = 0.009$) and to use a length of 17 ft. for the standard tunnel section. The final dimensions of the model are given on Fig. 2.

Tests on Model

The standard tunnel section of the model was constructed of galvanized iron sheets, the remainder, including the bridge piers, being of wood and all the surfaces over which water flowed being well varnished. The level floor of the approach channel was made of unplaned boards, and the curved surface connecting the spillway lip with this floor was built up with coarse sand mortar, intentionally roughened. It has been mentioned that in the design of the spiral approach for the full-sized spillway, the value of n in Kutter's formula was taken as 0.035. When the reduction in hydraulic radius of the model is taken into account, it is found that the surfaces of the spiral approach for the model must have a roughness corresponding to $n = 0.017$, in order to maintain the desired flow ratio, and, for this purpose, the vertical walls of the approach

channel were made of galvanized iron, coated with heavy paint into which coarse sand had been thrown before the paint dried. As mentioned previously, the spillway model proper was made as smooth as practicable, in order to attain a coefficient of roughness, $n = 0.009$, if possible, but it should be noted that failure to secure so low a coefficient is on the side of safety, as it would decrease the flow through the spillway model under the given heads and thus indicate conservative values of the capacity for the full-sized spillway under the corresponding heads. The general appearance of the model is shown in Fig. 16.

As the radial dimensions of the spiral approach channel for the clear crest type differed from those for the flash-board pier type, it was necessary to construct the model of the channel in such a way as to enable both channels to be tested. In order to facilitate the work of the tests, the model was constructed to permit the shift from one spiral channel to the other to be made very quickly. The flash-board piers were attached by dowels, and so could be placed or removed readily. The galvanized-iron strips used for the vertical walls of the approach channel were cut off on a slope, corresponding to the slope of the ground at the spillway site, to a point well above the crest of the model. The outstanding edges of the wooden floor of the approach channel were also sloped down for about 2 in. at an angle corresponding to the ground slope, in order to simulate the full-sized approach-channel conditions and to avoid the contraction that would be caused by an abrupt edge at this point. The position of the approach model was such that it bore the same relation to the tunnel model as that which obtains in the full-sized spillway.

As at first constructed, the pier models extended slightly beyond the theoretical down-stream edge of the converging flat crest weir section, but during the progress of the Whitingham tests the inner ends were cut off until the length of piers was only 4.8 in.

In all cases, the flow through the spillway model was measured by a sharp-edged weir, and the heads both on this measuring weir and on the crest of the spillway model were measured by hook-gauges located in stilling-boxes. Pressures within the spillway were measured in open-end, vertical, glass tubes attached by rubber tubing to small flexible copper tubing leading to the piezometer connections. Depths of water at the discharge end of the tunnel were determined by measuring with a rule from the crown of the tunnel to the water surface. Profiles for the water surface on the clear crest model were determined by measuring horizontally along a level rod (Figs. 7 and 8) fixed in place and then vertically downward. Some of the profiles in the case of the model of the pier crest type were taken by marking the water-line on the piers with a lead pencil. Studies to determine the best shape for the pier ends were made by observing the contraction conditions during the tests and by marking the lines of contraction on the flat surface of the spillway model between the piers.

Worcester Tests of June 7, 1922.—The purpose of the first two series of tests on the model, carried out at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, at Worcester, Mass., on June 7 and 9, 1922, was to ascertain first, whether the capacity of the spillway without a spiral approach channel agreed with the predicted capacity at the design head of 7 ft.;

and, second, whether the water would be likely to enter the upper end of the shaft in such a disturbed state of flow as to produce undue losses of head in the shaft and tunnel and thus possibly cause erosion of the concrete lining. As described previously, the curve of the spillway was designed so that water passing over the crest under the specified head of 7 ft., would adhere everywhere to the face of the spillway, as it was believed that this was a condition precedent to a small amount of disturbance. As experimental data available for fixing the profile of the spillway were rather meager, this point was carefully observed during the tests.

If the model proved satisfactory in regard to these two main objects, it was planned to obtain experimental profiles of the water surface for various heads on the crest of the model ranging from 0 in. to 5 in. (the latter depth corresponding to one of 15 ft. on the full-sized spillway), and to make observations on the pressure conditions within that part of the spillway above the 90° bend. Fortunately, all these projects were successfully undertaken.

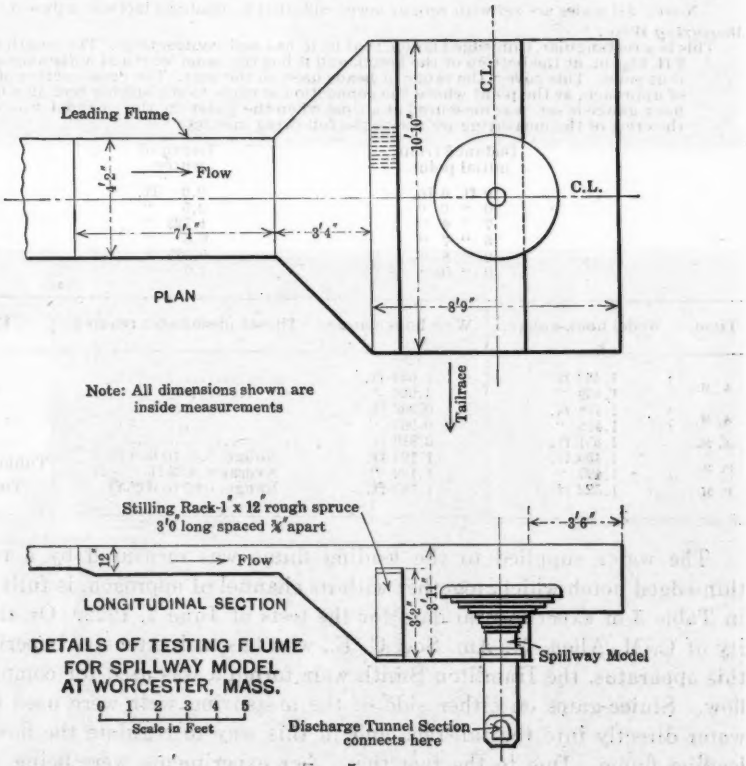

Fig. 5 shows a plan and longitudinal section of the testing flume used at Worcester, and a general view of the equipment looking up stream toward the leading flume is given in Fig. 6. It will be noted that there was a considerable drop in the floor between the two flumes, which caused the water to accelerate

as it entered the testing flume. This, together with the fact that the normal velocities in the leading flume were already rather high, led to a considerable disturbance of the water surface in the testing flume. An attempt was made to eliminate this disturbance by means of a wooden screen placed across the entrance to the flume and, although this proved beneficial, it did not eliminate the whole trouble. The disturbed condition of the water surface is clearly shown on the photographs, Figs. 7 and 8, taken during the tests, both by the irregularities or ripples and by the bubbles coming to the surface due to the entraining of air as the water passed through the stilling-rack, where it was violently agitated. Tests made on the flume indicated that it was sufficiently tight to justify neglecting leakage as a factor in the work.

TABLE 3.—EXPERIMENTAL DATA ON WORCESTER TESTS OF JUNE 7, 1922.

General:

Mean elevation of crest of spillway model = 1.281 ft. on model hook-gauge.
" " " " " measuring weir = 0.650 " " weir " "
Reading of 3.900 on scale of throat piezometer = reading of 2.000 on model hook-gauge.

NOTE: All scales are set with zero at lower end, that is, readings increase upward.

Measuring Weir:

This is a rectangular, thin-edged notch, that is, it has end contractions. The length of the weir is 3 ft. 6$\frac{3}{32}$ in. at the bottom of the notch, and it has the same length at a distance of 6 in. above that point. This covers the range of heads used on the weir. The cross-section of the channel of approach, at the point where the connection is made to the stilling-box, in which the weir hook-gauge is set, was measured at a time when the water in the channel was 0.2 ft. below the crest of the measuring weir with the following results:

Distance from initial point.	Depth of water.
10 ft. 6 in.	0.0 ft.
10 " 0 "	2.5 "
7 " 6 "	1.725 "
5 " 7 "	0.6 "
2 " 8 "	0.525 "
0 " 0 "	1.0 "

Time.	Model hook-gauge.	Weir hook-gauge.	Throat piezometer reading.	Remarks.
A. M.	1.450 ft. 1.439 "	1.044 ft. 1.026 "	....	.....
A. M.	1.418 ft. 1.416 "	0.969 ft. 0.967 "	....	.....
A. M.	1.351 ft.	0.810 ft.	.....	.....
P. M.	1.499 ft. 1.495 "	1.123 ft. 1.122 "	Range, 0.30 to 0.17 ft. Average, 0.22 ft.	Tunnel nearly full.
P. M.	1.522 ft.	1.160 ft.	Range, 0.05 to 0.08 ft.	Tunnel full.

The water supplied to the leading flume was measured by a rectilinear, thin-edged notch which, together with its channel of approach, is fully described in Table 3 of experimental data for the tests of June 7, 1922. On the authority of C. M. Allen, M. Am. Soc. C. E., who has had extensive experience with this apparatus, the Hamilton Smith weir formula was used for computing this flow. Sluice-gates on either side of the measuring weir were used to by-pass water directly into the tail-race and in this way to regulate the flow into the leading flume. Due to the fact that other experiments were being conducted on the same day, the discharge from which formed the supply for the spillway tests, difficulty was encountered in keeping a constant flow through the spillway model for any length of time, and, therefore, the number of observations was limited. Photographs were taken from a platform immediately over the center of the model (Fig. 6), the camera pointing vertically downward.

FIG. 7.—SPILLWAY MODEL IN WORCESTER FLUME UNDER DESIGN HEAD OF 2.32 INCHES.

FIG. 6.—VIEW OF TESTING FLUME AND SPILLWAY MODEL, WORCESTER, MASS.

FIG. 8.—SPILLWAY MODEL IN WORCESTER FLUME UNDER HEAD OF 10.25 INCHES, SHOWING VORTEX.

FIG. 9.—SPILLWAY MODEL WITH SPIRAL APPROACH CHANNEL IN WHITINGHAM, VT., FLUME, AT DISCHARGE OF 2.35 SECOND-FEET.

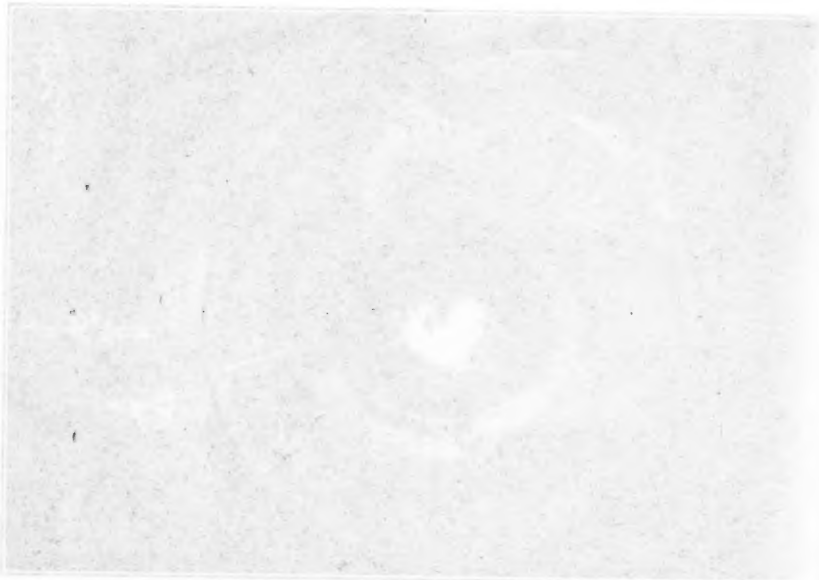

TABLE 4.—Analysis of Experimental Data on Worcester Tests of June 7, 1922.

(Length of Measuring Weir = 3.508 Ft.)

Length of crest of model, measured along up-stream edge = 13.963 ft.; B (for model) = 1.0625 ft.

Vacuum at Throat of Flaring Section (Piezometer No. 4):

Head on model.	Q.	Vacuum, in inches of water.
0.216 ft. = 2.59 in.	3.72 sec-ft.	9.76
0.241 " = 2.89 "	4.16 "	13.18

No.	Head on measuring weir, D, in feet.	Value of C.	Value of $D^{3/2}$.	Approximate value of Q.	Cross-sectional area of channel of approach, in square feet.	Approximate velocity of approach, in feet per second.	Velocity head h_v, in feet.	Head on model.	$H^{3/2}$.	Value of C for model.	Value of $\frac{H}{B}$ for model.
1	0.394	3.284	0.2474	2.85	18.3	0.156	0.0004	0.169 ft. = 2.03 in.	0.0695	2.94	0.159
2	0.376	3.290	0.2306	2.66	18.2	0.146	0.0003	0.158 " = 1.90 "	0.0628	3.04	0.149
3	0.319	3.310	0.1802	2.10	17.6	0.119	0.0002	0.137 " = 1.64 "	0.0508	2.96	0.129
4	0.317	3.311	0.1785	2.07	17.5	0.118	0.0002	0.135 " = 1.62 "	0.0496	2.99	0.127
5	0.150	3.416	0.0581	0.70	15.8	0.044	0.0000	0.070 " = 0.84 "	0.0185	2.71	0.066
6	0.473	3.264	0.3253	3.73	19 2	0.194	0.0006	0.218 " = 2.62 "	0.1018	2.63	0.205
7	0.472	3.265	0.3248	3.72	19.2	0.194	0.0006	0.214 " = 2.57 "	0.0990	2.69	0.202
8	0.510	3.256	0.3642	4.16	19.6	0.212	0.0007	0.241 " = 2.89 "	0.1183	2.52	0.226

Note.—The velocity heads of approach were found to be so small as to be negligible, hence the approximate values of Q are also the final values of Q. Values of C in the weir formula, $Q = CLH^{3/2}$ were taken from the Hamilton Smith tables (See *Water Supply Paper No. 200*, U. S. Geological Survey, p. 36.).

The experimental data for these tests and their analyses are shown in Tables 3 and 4, and, from these data, the capacity curve shown on Fig. 10 was plotted. This curve represents the capacity of the clear crest type of model set in deep water, that is, without a spiral approach channel. These first tests showed conclusively that the capacity of the spillway with a depth of 2.33 in. on the crest was close to the predicted capacity, being about 96% of that amount. The tests also indicated that this was not the maximum, but that, apparently, its practical limit was reached with a depth of about 2.89 in. on the crest, at which time the flow was about 20% greater than the nominal maximum capacity. It was thus seen that if the same increase were maintained in the full-sized spillway the maximum capacity would be about 32 000 sec-ft. and that the corresponding depth on the spillway would be about 8.67 ft,

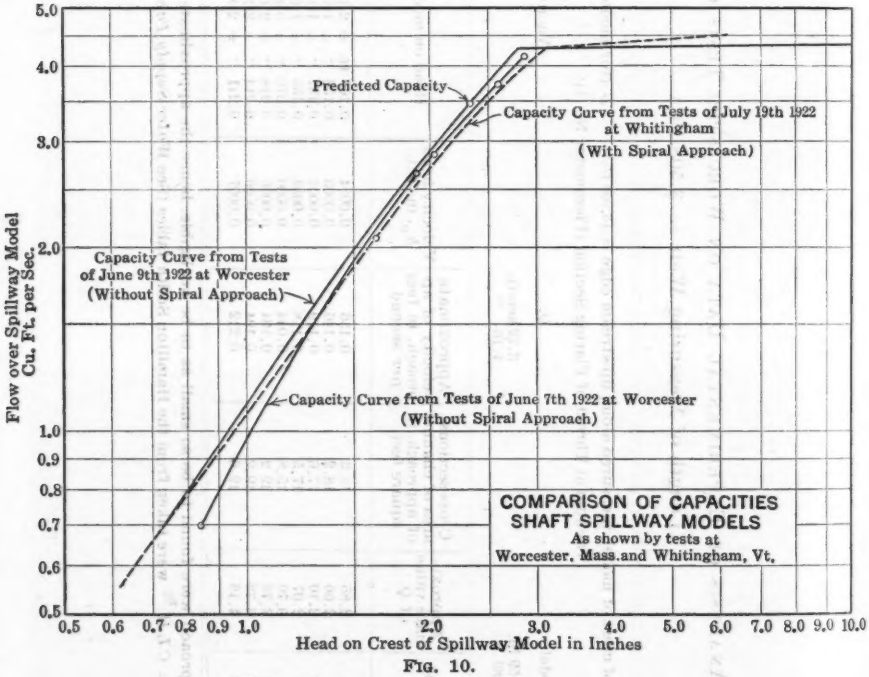
Predicted Capacity
Capacity Curve from Tests of July 19th 1922 at Whitingham (With Spiral Approach)
Capacity Curve from Tests of June 9th 1922 at Worcester (Without Spiral Approach)
Capacity Curve from Tests of June 7th 1922 at Worcester (Without Spiral Approach)
COMPARISON OF CAPACITIES SHAFT SPILLWAY MODELS
As shown by tests at Worcester, Mass. and Whitingham, Vt.
Flow over Spillway Model Cu. Ft. per Sec.
Head on Crest of Spillway Model in Inches
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.5 0.6 0.7 0.8 0.9 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

FIG. 10.

These first tests solved the problem as to the form of disturbance that would be caused by the loss of the horizontal velocity of the jet. At heads as low as 1.64 in. on the model, corresponding to 4.92 ft. on the full-sized spillway, the jet began to close in, forming a small disturbed "mushroom" far down in the throat of the spillway. This mushroom grew both in volume and height from that head, being very distinct from the flowing jet of water which could be seen to slide smoothly over the spillway model (see Figs. 7, 9, and 19), and under this disturbed part. At some head between 2.62 in. and 2.87 in., this distinction between the mushroom and the flowing jet began to disappear until, at the latter head, corresponding to the practical maximum capacity of the model, the spillway was entirely flooded, acting practically as a vertical orifice. As

the experimental capacity agreed so closely with the predicted capacity, it was evident that this mushroom of disturbed water had no back-water effect tending to decrease the discharge, but simply represented a volume of water maintained in violent motion by the energy continuously being lost in impact as the jet closes in on itself, destroying the horizontal velocity. The jet of water issuing from the end of the tunnel showed a comparatively small volume of entrained air, and the check on the predicted capacity proved that any air so entrained had only a negligible effect, if any, on the capacity.

These first tests also showed clearly that the water at no time left the surface of the spillway and that, therefore, the shape of the spillway was satisfactory from that standpoint. However, the one piezometer tube which had been placed at the upper end of the cylindrical section of the spillway shaft, and on which observations were made for the last three runs on June 7, indicated a vacuum of from 9.76 to 13.18 in. of water at this point, when the spillway was operating at or near its maximum capacity. As already stated under "Design", it was assumed that the pressure at this point, as well as that at all points above it, would be atmospheric. In view of the indications of the existence of a partial vacuum, it was decided to install four additional piezometer tubes before making further capacity tests and taking profiles of the water surface. The locations of these additional tubes, as well as the location of the original tube, are shown on Fig. 2.

For the purpose of comparison with the discharge coefficients for flat crest weirs shown on Fig. 4, the discharge coefficients for the spillway model were computed and are given in Table 4. It is seen that for heads less than the design head of 7 ft., that is, for ratios of $\frac{H}{B}$ less than 0.183, the model coefficients are much higher than those given by the curve, this difference being due to the sloping crest of the model which, for small values of $\frac{H}{B}$, has a slope such that it much more than compensates for the convergence. For values of $\frac{H}{B}$ greater than 0.183, the model coefficient begins to drop off rapidly as the spillway "floods out".

Worcester Tests of June 9, 1922.—Before proceeding with the second and last series of tests at Worcester, the additional piezometer tubes were installed. The flow conditions in the channel leading to the measuring weir were very bad on June 9, 1922, as the experiments previously mentioned were still in progress above this point, resulting in uneven conditions of flow. About all that could be done was to let the stream flow as it would, increasing from a small flow over the model to a point well above that at which the model floods out and then dropping again to almost zero flow. Very complete notes on flow conditions were made, and complete sets of pressure readings on all the piezometers were taken, as well as two profiles of the water surface over the model.

After the completion of this second set of tests, it was noted that the zero readings of the hook-gauge giving depths over the spillway model differed by

TABLE 5.—ANALYSIS OF EXPERIMENTAL DATA ON WORCESTER TESTS OF JUNE 9, 1922.

Time, P. M.	Head on measuring weir, D, in feet.	Hamilton Smith coefficient for measuring weir, C.	$D^{\frac{3}{2}}$.	Q, over measuring weir, in cubic feet per second.	Head on crest of spillway model, H, in feet.	$H^{\frac{3}{2}}$.	Computed coefficient for spillway model	Head, in feet, at piezometer connection:					Q_1, over spillway model (when different from Q).
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	
1:35	0.349	3.298	0.2062	2.38	0.143	0.0541	3.15	0.00	0.01	−0.03	−0.24	0.21	....
40	0.365	3.293	0.2206	2.54	0.149	0.0575	3.16	0.00	0.01	0.05	−0.24	0.20	....
45	0.377	3.289	0.2317	2.66	0.154	0.0605	3.15	0.00	0.02	0.01	−0.22	0.18	....
50	0.394	3.284	0.2474	2.84	0.165	0.0670	3.04	0.01	0.35	0.09	−0.31	0.17	....
55	0.410	3.279	0.2625	3.02	0.174	0.0726	2.98	0.05	0.44	0.09	−0.34	0.18	....
2:00	0.450	3.269	0.3019	3.46	0.194	0.0854	2.91	0.17	0.48	−0.23	−0.64	−0.45	....
05	.....	.....	.....	....	0.214	.....	....	0.24	0.47	−0.36	−0.89	−0.72	....
10	0.487	3.261	0.3398	3.88	0.212	0.0976	2.85	0.24	0.50	−0.40	−0.89	−0.67	....
15	0.488	3.261	0.3409	3.90	0.237	.....	....	0.28	0.49	−0.43	−0.88	−0.69	....
20	0.489	3.260	0.3420	3.90	0.214	0.0990	2.81	0.27	0.52	−0.41	−0.91	−0.67	....
25	0.510	3.256	0.3642	4.15	0.227	0.1082	2.74	0.42	0.57	−0.35	−1.29	−0.72	....
30	0.522	3.254	0.3772	4.30	0.253	0.1273	2.42	0.45	0.59	−0.40	−1.29	−1.02	....
33	0.533	3.252	0.3891	4.43	0.625	.....	....	0.87	0.95	−0.13	−1.14	−0.67	4.33
36	0.556	3.248	0.4146	4.71	0.771	.....	....	1.04	1.06	−0.03	−1.21	−0.42	4.34
38	0.567	3.247	0.4269	4.86	0.917	.....	....	1.15	1.10	0.05	−1.09	−0.42	4.34
42	0.584	3.244	0.4463	5.07	1.279	.....	....	1.71	1.63	0.42	−0.99	−0.67	4.35
55	0.636	3.237	0.5072	5.75	1.49	.....	....	1.72	1.70	0.32	−0.97	−0.25	4.35
3:03	0.633	3.230	0.5644	6.37	1.70	.....	....	1.92	1.85	0.49	−0.95	−0.27	4.35
05	0.646	3.235	0.5192	5.88	1.62	.....	....	1 84	1.76	0.37	−1.01	−0.25	4.35
07	0.563	3.248	0.4225	4.81	1.50	.....	....	1.67	1.65	0.23	−0.99	−0.30	4.35
08	.....	.....	.....	....	1.42	.....	....	....	....	....	....	....	4.35
10	0.537	3.252	0.3935	4.48	1.33	.....	....	1.60	1.64	0.52	−0.66	−0.37	4.35
13	0.526	3.253	0.3815	4.35	1.35	.....	....	1.59	1.64	0.52	−0.69	−0.62	4.35
16	0.496	3.259	0.3494	3.99	1.27	.....	....	1.50	1.58	0.65	−0.57	−0.22	4.35
18½	0.463	3.266	0.3151	3.61	0.833	.....	....	0.94	1.10	0.22	....	−1.02	4.34

TABLE 5.—(*Continued.*)

Time, P. M.	Head on measuring weir, D, in feet.	Hamilton Smith coefficient for measuring weir, C.	$D^{\frac{3}{2}}$.	Q, over measuring weir, in cubic feet per second.	Head on crest of spillway model, H, in feet.	$H^{\frac{3}{2}}$.	Computed coefficient for spillway model.	Head, in Feet, at Piezometer Connection:					Q_1, over spillway model (when different from Q).
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	
3:20	0.460	3.266	0.3120	3.58	0.451	.....	....	0.54	0.74	0.07	—0.54	—0.82	4.32
21½	0.453	3.268	0.3049	3.50	0.229	0.1096	....	0.42	0.60	—0.03	—0.69	—0.72	4.15
23	0.449	3.269	0.3009	3.46	0.207	0.0942	....	0.27	0.53	—0.11	—0.59	—0.62	3.72
25	0.442	3.271	0.2939	3.37	0.194	0.0854	....	0.30	0.55	—0.09	—0.54	—0.62	3.48
28	0.432	3.273	0.2840	3.26	0.189	0.0822	....	0.22	0.52	0.00	—0.44	—0.47	3.32
31	0.420	3.276	0.2722	3.12	0.181	0.0770	....	0.17	0.53	0.09	—0.32	—0.37	3.17
34	0.409	3.279	0.2616	3.01	0.175	0.0732	2.95	0.12	0.52	0.11	—0.29	—0.20	....
37	0.402	3.281	0.2549	2.94	0.173	0.0720	2.92	0.04	0.48	0.07	—0.26	—0.12	....
40	0.407	3.280	0.2596	2.98	0.175	0.0732	2.92	0.04	0.46	—0.01	—0.29	—0.17	....
43	0.222	3.359	0.1046	1.23	0.087	0.0257	3.43	0.00	0.00	—0.05	—0.14	0.13	....
46	0.271	3.330	0.1411	1.65	0.110	0.0365	3.24	0.00	0.00	—0.06	—0.13	0.13	....
49	0.245	3.350	0.1140	1.34	0.095	0.0293	3.28	0.00	—0.08	—0.05	—0.10	0.11	....
52	0.212	3.364	0.0976	1.15	0.089	0.0266	3.10	0.00	—0.08	—0.04	—0.16	0.13	....
55	0.197	3.377	0.0874	1.03	0.079	0.0222	3.32	0.00	—0.10	—0.04	—0.14	0.13	....
58	0.250	3.341	0.1250	1.47	0.104	0.0336	3.14	0.00	—0.08	—0.05	—0.11	0.16	....
4:01	0.161	3.407	0.0645	0.77	0.065	0.0166	3.32	0.00	—0.08	—0.02	—0.18	0.11	....
04	0.160	3.408	0.0640	0.76	0.064	0.0162	3.36	0.00	—0.08	—0.04	—0.13	0.10	....
07	0.162	3.407	0.0652	0.78	0.065	0.0166	3.36	0.00	—0.08	—0.04	—0.15	0.08	....
10	0.158	3.410	0.0628	0.75	0.065	0.0166	3.23	0.01	—0.08	—0.03	—0.14	0.08	....
13	0.154	3.413	0.0605	0.72	0.063	0.0158	3.26	0.00	—0.08	—0.04	—0.16	0.09	....
16	0.156	3.411	0.0616	0.74	0.068	0.0158	3.36	0.00	—0.08	—0.04	—0.15	0.08	....
19	0.160	3.408	0.0640	0.76	0.065	0.0166	3.28	0.00	—0.08	—0.04	—0.11	0.09	....
22	0.156	3.411	0.0616	0.74	0.064	0.0162	3.27	0.00	—0.08	—0.03	—0.12	0.08	....
25	0.154	3.413	0.0605	0.72	0.062	0.0155	3.32	0.00	—0.08	—0.04	—0.16	0.11	....
28	0.158	3.410	0.0628	0.75	0.064	0.0162	3.31	0.00	—0.08	—0.04	—0.16	0.11	....
30	0.155	3.412	0.0610	0.73	0.064	0.0162	3.23	0.00	—0.08	—0.03	—0.12	0.10	....

0.02 ft. from the zero readings taken at the beginning of this day's experiments. The cause of this difference was carefully sought, and it was decided that a change in zero probably occurred between 2:38 and 2:42 P. M., on the afternoon of June 9, when the spillway model was flooded out, and a rule was rested on the point of the hook in order to continue the readings of the water surface. The experimental data were reduced, therefore, on this basis.

The readings for these tests are not given, but Table 5, showing the analysis of the experiments, is included. The results of these data, as far as the capacity of and the pressure within the model are concerned, are plotted in the form of curves on Figs. 11 and 12. For flows over the spillway model in excess of 3 sec-ft., the data are unreliable, due to the effect of storage in the testing and leading flumes. On account of this storage, the flow over the spillway model does not correspond to the flow over the measuring weir at the same instant. However, from a careful study of the data, it was possible to plot a fairly reliable capacity curve for heads on the crest of the model of from 0.75 in. to 16 in.

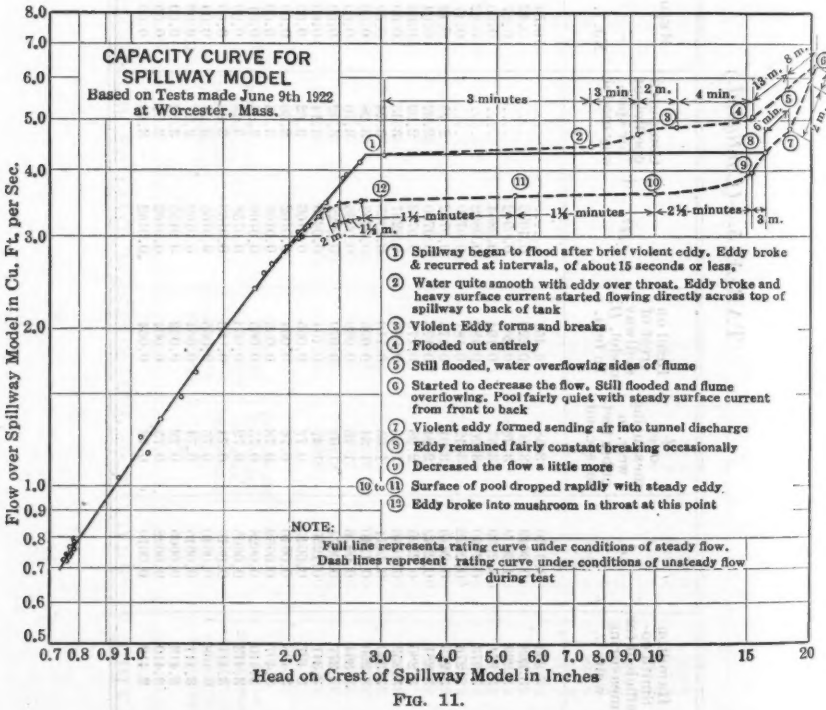
CAPACITY CURVE FOR SPILLWAY MODEL
Based on Tests made June 9th 1922 at Worcester, Mass.
Flow over Spillway Model in Cu. Ft. per Sec.
Head on Crest of Spillway Model in Inches
3 minutes
3 min.
2 m.
4 min.
12 m.
8 m.
6 min.
2 m.
12 m.
1½ minutes
1½ minutes
2½ minutes
3 m.
2 m.
1½ m.
(1) Spillway began to flood after brief violent eddy. Eddy broke & recurred at intervals, of about 15 seconds or less.
(2) Water quite smooth with eddy over throat. Eddy broke and heavy surface current started flowing directly across top of spillway to back of tank
(3) Violent Eddy forms and breaks
(4) Flooded out entirely
(5) Still flooded, water overflowing sides of flume
(6) Started to decrease the flow. Still flooded and flume overflowing. Pool fairly quiet with steady surface current from front to back
(7) Violent eddy formed sending air into tunnel discharge
(8) Eddy remained fairly constant breaking occasionally
(9) Decreased the flow a little more
(10) to (11) Surface of pool dropped rapidly with steady eddy
(12) Eddy broke into mushroom in throat at this point
NOTE:
Full line represents rating curve under conditions of steady flow.
Dash lines represent rating curve under conditions of unsteady flow during test

FIG. 11.

The capacity curve deduced from the tests of June 9 was also plotted on Fig. 10 for the purpose of comparing it with the capacity curve deduced from the tests of June 7. It is to be noted that the two curves are in fair agreement, and that the curve based on the test of June 9 passes exactly through the point of predicted capacity. The pressure readings, reduced to heads above or below atmospheric pressure, expressed in feet of water, have been plotted on

Fig. 12, both with the flow over the model, in cubic feet per second, and with the depth of water on crest of the model, as abscissa. It will be noted that the tests of June 9 were confirmatory of those of June 7 in so far as the existence of a vacuum at Piezometer No. 4 is concerned, and that they also showed a vacuum to exist at Piezometers Nos. 3 and 5 for certain stages of flow. The

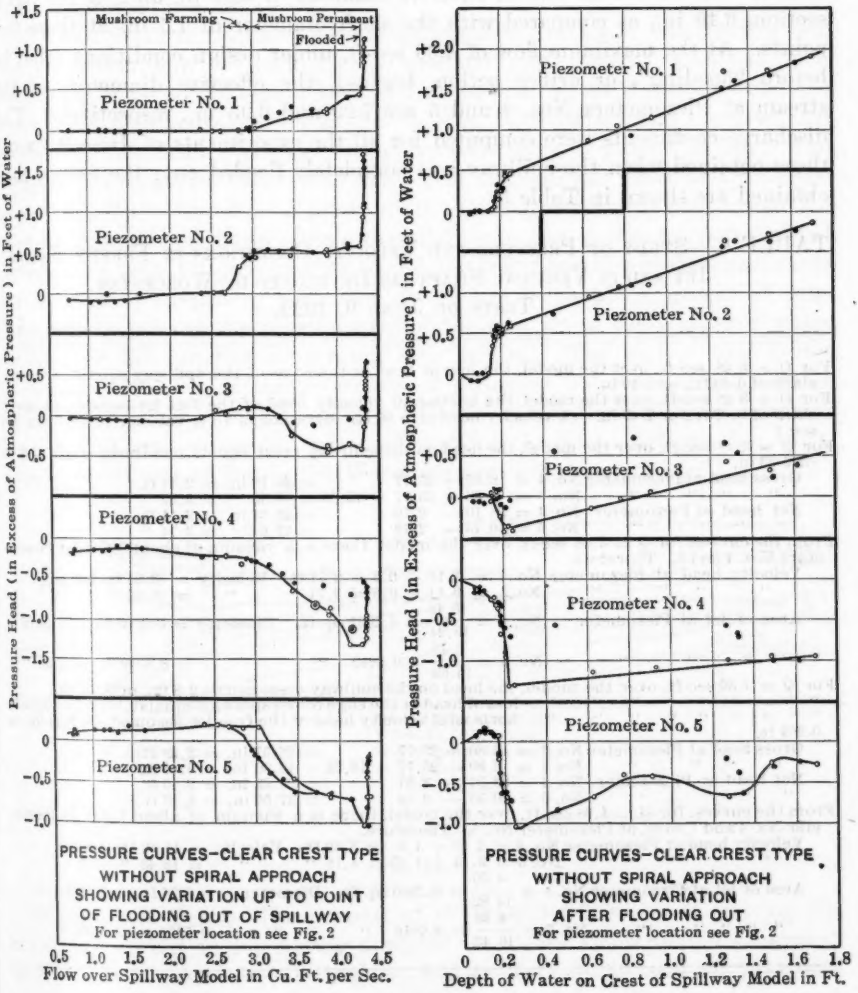
Mushroom Farming
Mushroom Permanent
Flooded
Piezometer No. 1
Piezometer No. 2
Piezometer No. 3
Piezometer No. 4
Piezometer No. 5
Pressure Head (in Excess of Atmospheric Pressure) in Feet of Water
PRESSURE CURVES—CLEAR CREST TYPE
WITHOUT SPIRAL APPROACH
SHOWING VARIATION UP TO POINT
OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
Flow over Spillway Model in Cu. Ft. per Sec.
PRESSURE CURVES—CLEAR CREST TYPE
WITHOUT SPIRAL APPROACH
SHOWING VARIATION
AFTER FLOODING OUT
For piezometer location see Fig. 2
Depth of Water on Crest of Spillway Model in Ft.

FIG. 12.

curves show that the amount of vacuum is very small until a flow of from 2.5 to 3.5 sec-ft. is reached, at which time the pressure conditions seem to change rapidly. The vacuums reach maximum values at about the time the model floods out and begins to act as a vertical orifice. It will be noted from Fig. 12 that the pressure rise after the model is flooded follows, in general, a straight line, which seems reasonable for the orifice condition which then obtains.

From the pressure curves of Fig. 12, a study the details of which are given in Table 6, was made of the natural Venturi tube effect which exists in the freely falling jet and in the vertical shaft at certain stages of the flow. The computations were made for the sections of the stream of water at Piezometers Nos. 4 and 5 and indicate that, at the former section, the stream corresponding to a flow of 3.48 sec-ft. has an effective diameter of 6.94 in. and, at the latter section, 6.40 in., as compared with the shaft diameter of 7.5 in. at these two points. At the maximum flow of 4.30 sec-ft. under design conditions (that is, before "flooding", or orifice action, begins), the effective diameters of the stream at Piezometers Nos. 4 and 5 are 7.29 and 6.93 in., respectively. The discharge coefficients were computed for all the experiments of June 9, except those obtained when the spillway was completely flooded out; the values thus obtained are shown in Table 5.

TABLE 6.—STUDY OF PRESSURE AND VELOCITY CONDITIONS IN FREELY FALLING JET AND IN VERTICAL SHAFT, AS DEVELOPED BY WORCESTER TESTS OF JUNE 9, 1922.

For Q = 3.48 sec-ft. over the model, the loss of head at the edge of the spillway equals one-thirty-sixth of 0.30 ft., or 0.10 in.
For Q = 3.48 sec-ft. over the model, the horizontal velocity head of the free jet equals one-thirty-sixth of 8.07 ft., or 2.69 in. (A velocity head of 8.07 ft. corresponds to a velocity of 22.75 ft. per sec.)
For Q = 3.48 sec-ft. over the model, the head on the spillway crest equals one-thirty-sixth of 7 ft. or 2.33 in.

Gross head at Piezometer No. 4 = 2.33 + 25.77 = 28.10 in. = 2.34 ft.
" " " " No. 5 = 2.33 + 25.77 + 12.33 = 40.43 " = 3.37 "
Net head at Piezometer No. 4 = 28.10 − 2.79 = 25.31 in. = 2.11 ft.
" " " " No. 5 = 40.43 − 2.79 = 37.64 " = 3.14 "

From the curves, for Q = 3.48 sec-ft. over the model, there is a vacuum of about 0.6 ft. at Piezometers Nos. 4 and 5. Therefore,

Velocity head at Piezometer No. 4 = 2.11 + 0.6 = 2.71 ft. Velocity = 13.24 ft. per sec.
" " " " No. 5 = 3.14 + 0.6 = 3.74 " " = 15.53 " "

Area of jet at Piezometer No. 4 = $\frac{3.48}{13.24}$ = 0.2627 sq. ft. Diameter = 0.5783 ft. = 6.94 in.

" " " " " No. 5 = $\frac{3.48}{15.53}$ = 0.2240 " " = 0.5340 " = 6.40 "

For Q = 4.30 sec-ft. over the model, the head on the spillway crest equals 2.8 in., or 0.233 ft.
" " = " " " " " " loss of head at the edge of the spillway equals 0.115 in., or 0.010 ft.
" " = " " " " " " horizontal velocity head of the free jet assumed as 3.23 in., or 0.269 ft.

Gross head at Piezometer No. 4 = 2.80 + 25.77 = 28.57 in. = 2.38 ft.
" " " " No. 5 = 2.80 + 25.77 + 12.33 = 40.90 in. = 3.41 ft.
Net head at Piezometer No. 4 = 28.57 − 3.34 = 25.23 in. = 2.10 ft.
" " " " No. 5 = 40.90 − 3.34 = 37.56 in. = 3.13 ft.

From the curves, for Q = 4.30 sec-ft. over the model, there is a vacuum of about 1.3 ft. at Piezometer No. 4 and 1.05 ft. at Piezometer No. 5. Therefore,

Velocity head at Piezometer No. 4 = 2.10 + 1.3 = 3.40 ft. Velocity = 14.82 ft. per sec.
" " " " No. 5 = 3.13 + 1.05 = 4.18 " " = 16.43 " "

Area of jet at Piezometer No. 4 = $\frac{4.30}{14.82}$ = 0.2900 sq. ft. Diameter = 0.6076 ft. = 7.29 in.

" " " " " No. 5 = $\frac{4.30}{16.43}$ = 0.2616 " " = 0.5772 " = 6.93 "

The photographs taken on June 9, 1922, as regards heads up to 2.72 in. on the crest of the model, are merely confirmatory of conditions as developed on June 7. The photographs as well as the discharge data for greater heads are merely matters of curious information, as a head of 4.33 in. on the model would correspond to a water surface at the crest of the earth dam. It is of interest to note that a vortex forms at a head of 9.25 in. on the model and is maintained in violent motion up to the highest head reached, namely, 16.42 in.,

which would correspond to nearly 50 ft. on the full-sized spillway. The capacity of the model, however, increases very slowly after the model floods out and begins to act as an orifice. This is to be expected, as the discharge then varies approximately as the square root of the total difference in elevation between head-water and tail-water, and any rise in head-water creates a small increase in the total head available.

It was difficult to maintain a constant flow long enough to take profiles and, for this reason, only two of any value were obtained. One of these was for a head fairly close to the design head of 7 ft. on the full-sized spillway, and the profile thus obtained has been plotted on Fig. 13, together with the predicted profile for that head. The agreement between the two profiles is fair. The greatest discrepancies occur near the lip of the spillway crest and near the upper end of the free jet. A study of the Cornell experiments on which the predicted profile was based, shows that the particular profile selected differed considerably from the ones just above and just below and this probably accounts for the first discrepancy; the other divergence is simply a matter of misjudgment in joining the upper nappe of the free jet to the computed water surface for the circular converging weir section. The highest and lowest elevations of the top of the mushroom of disturbed water are shown on this profile. The lower observed profile is also shown on Fig. 13, as a matter of interest, although no profile was predicted for the flow corresponding to it. Typical conditions of flow in the Worcester tests are shown in Fig. 7.

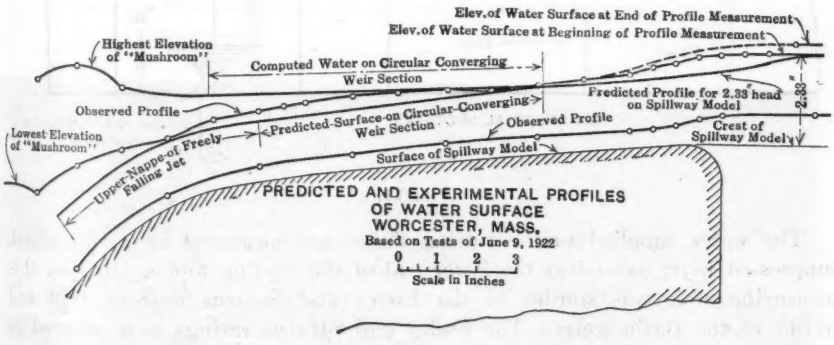
Elev. of Water Surface at End of Profile Measurement
Elev. of Water Surface at Beginning of Profile Measurement
Highest Elevation of "Mushroom"
Computed Water on Circular Converging Weir Section
Observed Profile
Predicted Profile for 2.33 head on Spillway Model
2.33
Predicted Surface on Circular Converging Weir Section
Observed Profile
Lowest Elevation of "Mushroom"
Crest of Spillway Model
Upper Nappe of Freely Falling Jet
Surface of Spillway Model
PREDICTED AND EXPERIMENTAL PROFILES OF WATER SURFACE WORCESTER, MASS.
Based on Tests of June 9, 1922
0 1 2 3
Scale in Inches

FIG. 13.

Whitingham Tests.—The purpose of the second two series of tests on the model, carried out in the testing flume of the Power Construction Company, on Sadawga Brook, near Whitingham, Vt., on July 19, 20, and 21, 1922, was to ascertain, first, the effect of the spiral approach on the spillway capacity and on the flow conditions; second, the effect of flash-board piers on the spillway capacity and flow conditions; and, third, the proper length of the flash-board piers and the best shapes for the ends of these piers.

An old crib dam on Sadawga Brook had been rebuilt to divert water into a leading flume which connects with the testing flume, shown in detail on Figs. 14, 15, and 16. Benefiting by the experience obtained at Worcester, the approach to the testing flume had been carefully designed with screens, parti-

tions, and movable baffles, so that the water entered the flume uniformly over its entire cross-section and without any disturbance whatever. Leakage tests on the flume indicated that it was very tight and justified the neglect of this factor in the tests.

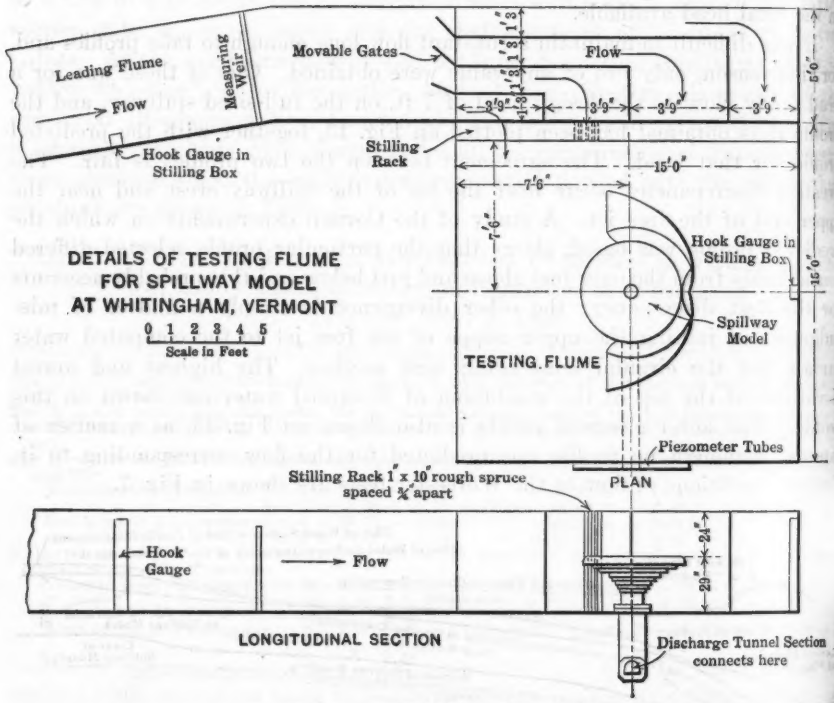

FIG. 14.

The water supplied to the testing flume was measured by a thin-edged, suppressed weir, located at the lower end of the leading flume. In size, this measuring weir was similar to the Fteley and Stearns weir of 1879 and to one of the Bazin weirs. The Fteley and Stearns ratings were adopted in reducing the experimental data for these tests.

The natural flow of Sadawga Brook on the days of the experiments was just about equal to the maximum capacity of the model and was quite uniform, so that it was possible to obtain a constant flow for any length of time desired. The flow into the leading flume was regulated by sluice-gates in the dam and a head-gate at the upper end of the flume.

A very complete series of photographs was taken from a platform immediately over the center of the model, the camera pointing vertically downward, of which Figs. 9, 19, 21 and 22 are typical. The supports for this platform are shown on Fig. 16. In order to obtain better illumination, sunlight was reflected into the throat by two large mirrors.

The experimental data for these tests are not given, but the analyses of the experiments are given in Tables 7 and 8. The results of these tests, as far as

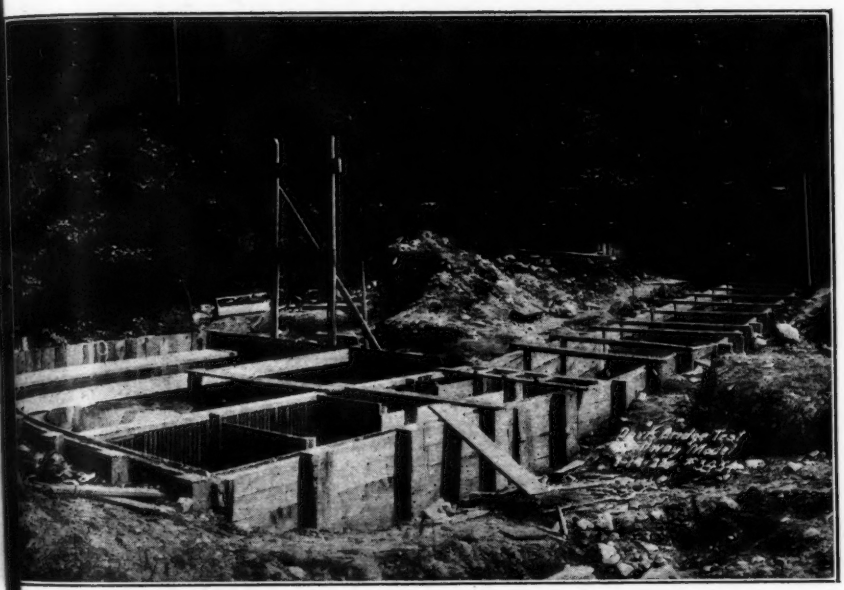

FIG. 15.—VIEW OF TESTING AND LEADING FLUMES, WHITINGHAM, VT.

FIG. 16.—VIEW OF TESTING FLUME AND MODEL, WHITINGHAM, VT.

TABLE 7.—ANALYSIS OF EXPERIMENTAL DATA ON WHITINGHAM TESTS OF JULY 19, 1922, ON CLEAR CREST MODEL.

No.	Head on measuring weir, D, in feet.	Head on crest of spillway model, in feet.	Q, over measuring weir, in cubic feet per second.	Depth of water surface below top of tunnel model, in feet.	Area of waterway, in square feet.	Velocity at end of tunnel, in feet per second.	Depth of water in tunnel, depth of tunnel = 0.60 ft., in feet.	Head in Feet, at Piezometer Connection:				
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
1	0.099	0.051	0.55	0.48	0.068	8.08	0.12	+0.01	+0.02	−0.02	−0.01	+0.02
2	0.131	0.069	0.83	0.43	0.100	8.30	0.17	+0.01	+0.02	−0.03	−0.02	+0.10
3	0.182	0.097	1.33	0.36	0.144	9.23	0.24	+0.00	+0.02	−0.05	−0.02	+0.18
4	0.204	0.111	1.58	0.315	0.172	9.18	0.285	+0.00	+0.02	−0.06	−0.01	+0.19
5	0.244	0.134	2.05	0.27	0.198	10.36	0.33	+0.00	+0.02	−0.04	+0.02	+0.14
6	0.267	0.149	2.34	0.25	0.210	11.15	0.35	+0.00	+0.03	+0.14(?)	−0.02	+0.14
7	0.305	0.174	2.86	0.185	0.244	11.72	0.415	+0.01	+0.21	−0.10	−0.26	−0.02
8	0.332	0.192	3.23	0.14	0.265	12.18	0.46	+0.04	+0.35	−0.20	−0.45	−0.23
9	0.365	0.219	3.71	0.025	0.302	12.28	0.575	+0.26	+0.51	−0.34	−0.67	−0.36
10	0.388	0.244	4.09	0.00	0.306	13.35	0.60	+0.40	+0.59	−0.30	−0.76	−0.36
11	0.405	0.267	4.35	0.00	0.306	14.22	0.60	+0.50	+0.63	−0.38	−0.82	−0.26
12	0.415	0.519	4.50	0.00	0.306	14.70	0.60	+0.76	+0.86	−0.24	−0.76	−0.21
13	0.411	0.509	4.45	0.00	0.306	14.54	0.60	+0.76	+0.84	−0.25	−0.77	−0.22

TABLE 8.—ANALYSIS OF EXPERIMENTAL DATA ON WHITINGHAM TESTS OF JULY 21, 1922, ON FLASH-BOARD PIER TYPE OF CREST MODEL.

No.	Head on measuring weir, D, in feet.	Head on crest of spillway model, in feet.	Q, over measuring weir, in cubic feet per second.	Depth of water surface below top of tunnel model, in feet.	Area of waterway, in square feet.	Velocity at end of tunnel, in feet per second.	Depth of water in tunnel, depth of tunnel = 0.60 ft., in feet.	Head, in Feet, at Piezometer Connection:				
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.*
1	0.174	0.099	1.25	0.38	0.132	9.47	0.22	+0.00	+0.02	−0.01	−0.02	−0.24
2	0.073	0.041	0.35	0.48	0.068	5.15	0.12	+0.01	+0.02	−0.00	−0.01	−0.23
3	0.133	0.074	0.85	0.425	0.103	8.25	0.175	+0.01	+0.02	−0.01	−0.02	−0.23
4	0.205	0.120	1.59	0.325	0.166	9.57	0.275	+0.00	+0.02	−0.02	−0.02	−0.23
5	0.251	0.150	2.14	0.285	0.190	11.26	0.315	0.00	+0.02	−0.02	−0.02	−0.23
6	0.265	0.160	2.32	0.255	0.206	11.26	0.345	0.00	+0.02	−0.04	−0.02	−0.23
7	0.306	0.188	2.86	0.195	0.239	11.96	0.405	0.00	+0.02	−0.08	−0.07	−0.23
8	0.330	0.205	3.21	0.15	0.260	12.35	0.45	−0.00	+0.01	−0.10	−0.14	−0.23
9	0.361	0.228	3.67	0.095	0.282	13.00	0.505	−0.02	+0.01	−0.22	−0.38	−0.23
10	0.388	0.248	4.09	0.02	0.303	13.48	0.58	−0.01	+0.24	−0.40	−0.68	−0.23
11	0.399	0.257	4.24	0.00	0.306	13.85	0.60	+0.42	+0.56	−0.40	−0.77	−0.23
12	0.388	0.250	4.09	0.02	0.303	13.48	0.58	−0.01	+0.18	−0.40	−0.96(?)	−0.86
13	0.406	0.325	4.35	0.00	0.306	14.22	0.60	+0.56	+0.67	−0.33	−0.74	−0.20
14	0.386	0.247	4.05	0.03	0.301	13.45	0.57	−0.00	+0.21	−0.40	−0.66	−0.94
15	0.365	0.229	3.71	0.095	0.282	13.16	0.505	−0.00	+0.02	−0.22	−0.40	−0.87
16	0.341	0.213	3.36	0.11	0.277	12.13	0.49	−0.00	−0.01	−0.12	−0.18	−0.86
17	0.312	0.193	2.94	0.20	0.236	12.45	0.40	0.00	+0.02	−0.06	−0.06	−0.86
18	0.277	0.166	2.46	0.235	0.218	11.27	0.365	+0.00	+0.02	−0.04	−0.02	−0.85
19	0.228	0.134	1.85	0.305	0.178	10.38	0.295	+0.01	+0.02	−0.02	−0.02	−0.85
20	0.205	0.118	1.59	0.335	0.159	10.00	0.265	+0.02	+0.02	−0.02	−0.01	−0.84
21	0.174	0.099	1.25	0.38	0.132	9.47	0.22	+0.01	+0.02	−0.02	−0.01	−0.84
22	0.132	0.071	0.84	0.43	0.100	8.40	0.17	+0.01	+0.02	−0.01	−0.02	−0.84

* Readings on this piezometer are unreliable.

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Flow over Spillway Model

capacity of and pressure within the model are concerned, are plotted in the form of curves on Figs. 17 and 18. The capacity curve deduced from the experiments of July 19 for the clear crest model with a spiral approach has also been plotted on Fig. 10 for the purpose of comparing it with the capacity curves deduced from the Worcester tests on June 7 and 9, 1922. It is to be noted that the capacity curves for the tests of June 9 and July 19 are in very good agreement for flows up to 0.8 cu. ft. per sec., but that, for greater flows, the July 19 curve is the lower; for flows greater than 1.5 cu. ft. per sec., the July 19 curve also falls below the June 7 curve. This reduction in capacity below

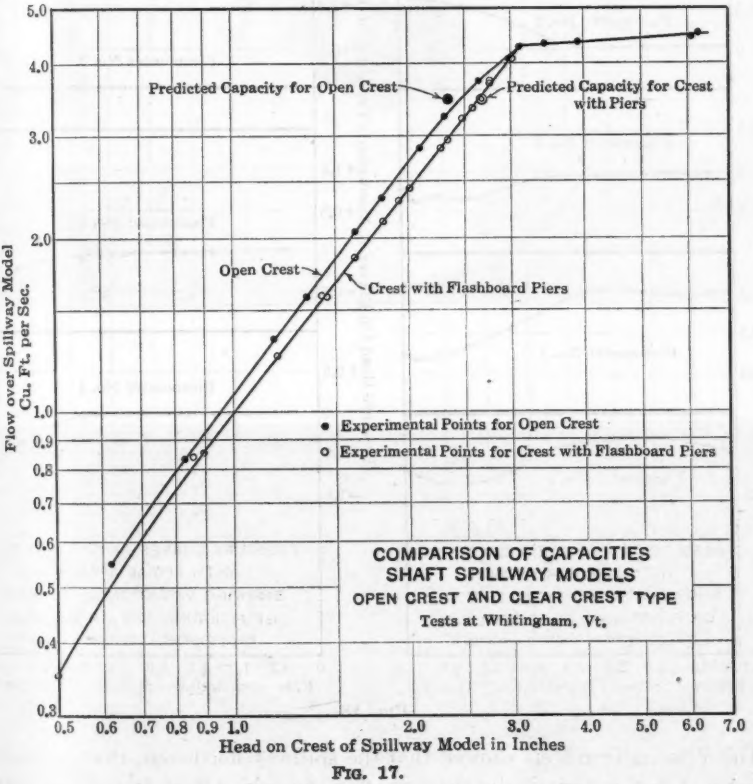


FIG. 17.

that obtained at Worcester is due undoubtedly to losses of head in the spiral approach and at the lip of the spillway, caused by the tangential velocity of the water. This belief is amply confirmed by the photographs taken in conjunction with the capacity tests for the clear crest model. (Figs. 9 and 19.) For flows over the model in excess of 1.34 cu. ft. per sec., a distinct projecting jet of water forms opposite the nose of the spiral approach channel, which is unquestionably due to the tangential velocity of the water in that channel. This projecting jet, shown on Fig. 9, undoubtedly represents a considerable loss of energy and accounts in large measure for the decrease in capacity previously mentioned. On many of the photographs (see Fig. 19), the spiral motion of the water is

indicated by the natural stream lines, and this is still more clearly shown on those taken during the stream-line tests (see Figs. 20 and 24), which were made by throwing shavings and sawdust into the flume and taking time exposures.

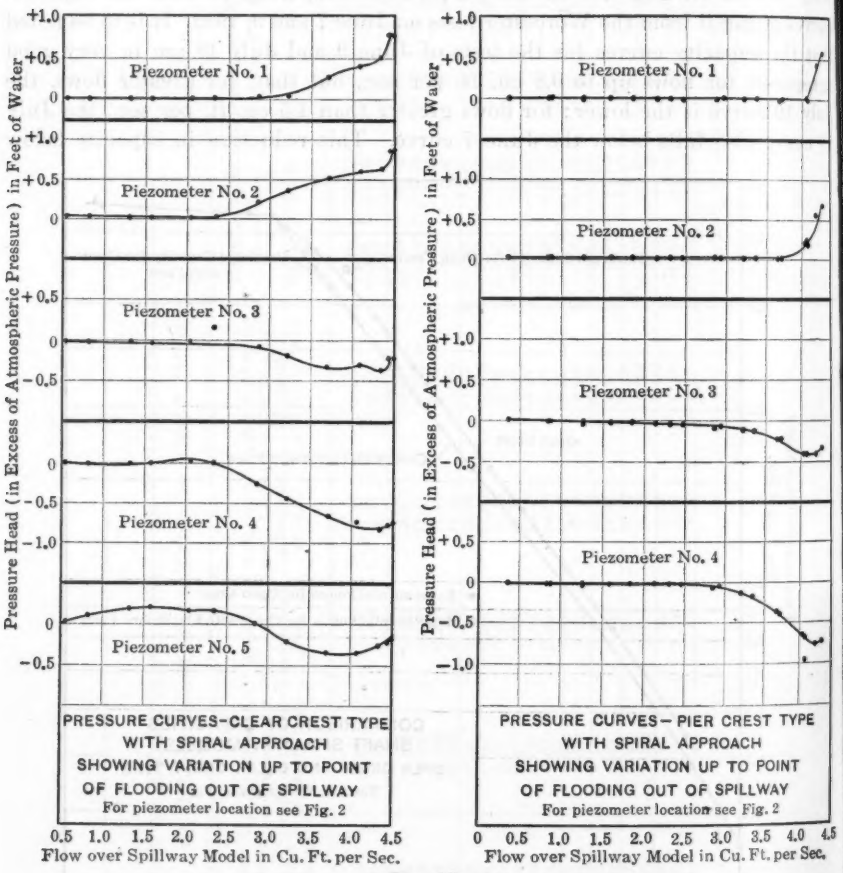

Pressure Head (in Excess of Atmospheric Pressure) in Feet of Water
+1.0
+0.5
0
Piezometer No. 1
Piezometer No. 2
Piezometer No. 3
Piezometer No. 4
Piezometer No. 5
−0.5
−1.0
PRESSURE CURVES—CLEAR CREST TYPE
WITH SPIRAL APPROACH
SHOWING VARIATION UP TO POINT
OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5
Flow over Spillway Model in Cu. Ft. per Sec.
PRESSURE CURVES—PIER CREST TYPE
WITH SPIRAL APPROACH
SHOWING VARIATION UP TO POINT
OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5
Flow over Spillway Model in Cu. Ft. per Sec.

FIG. 18.

The Whitingham tests showed that the spillway floods out, that is, begins to act as an orifice, at practically the same flow, namely, 4.3 cu. ft. per sec., whether the model is provided with a spiral approach or set in open water, and whether it is provided with the flash-board piers or left with an open crest. This is as it should be, as the point at which flooding out begins is determined by the tunnel and shaft and not by conditions at the top of the spillway. The head at which the spillway floods out, however, should be determined by crest conditions, and the experiments show this to be the case.

The experimental capacity curve for the flash-board pier type of crest (Fig. 17) passes almost exactly through the point of predicted capacity for a head of 8 ft. on the full-sized spillway. This indicates that the design assumptions as to the loss of head in the spiral approach channel, at the lip of the spillway and

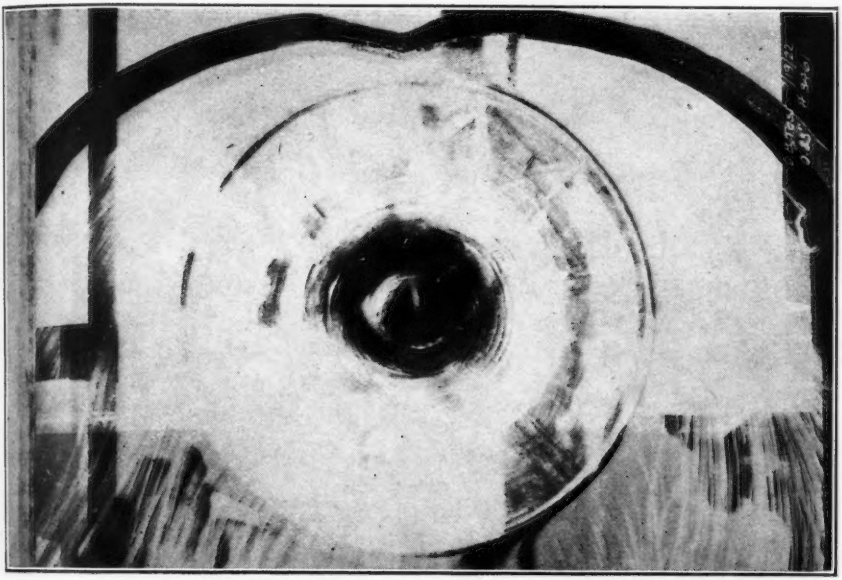


FIG. 20.—TYPICAL STREAM-LINE PHOTOGRAPH WITHOUT PIERS.

FIG. 19.—SPILLWAY MODEL WITH SPIRAL APPROACH CHANNEL AT DISCHARGE OF 4.09 SECOND-FEET, SHOWING RIDGE OF WATER OPPOSITE POINT OF SPIRAL.

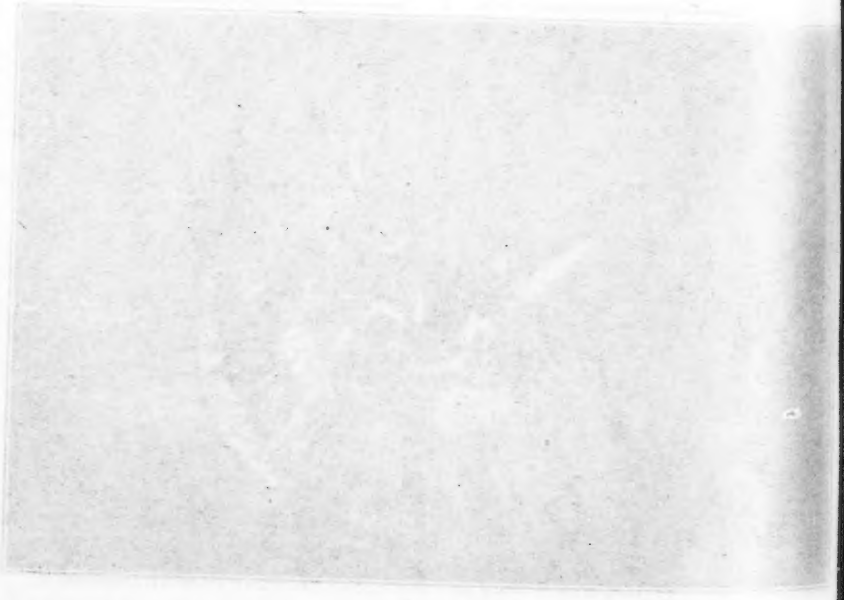


FIG. 21.—SPILLWAY MODEL WITH SPIRAL APPROACH AND PIERS AT DISCHARGE OF 2.43 SECOND-FEET.

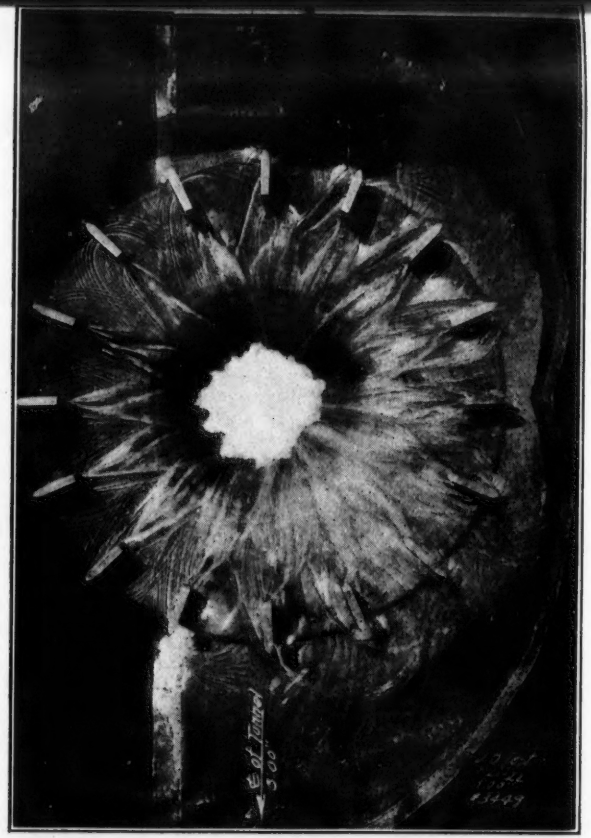
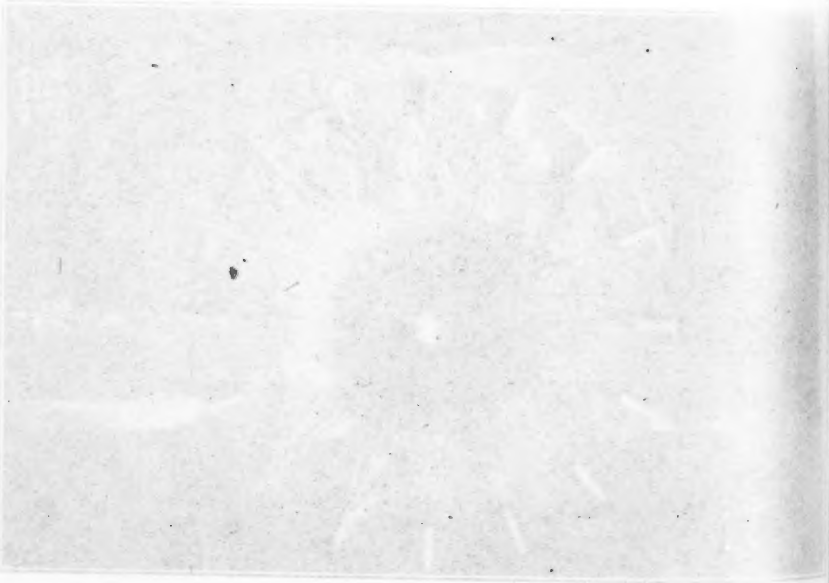


FIG. 22.—SPILLWAY MODEL WITH SPIRAL APPROACH AND PIERS AT DISCHARGE OF 4.10 SECOND-FEET, SHOWING EFFECT OF PIERS ON IMPROVING FLOW CONDITIONS.



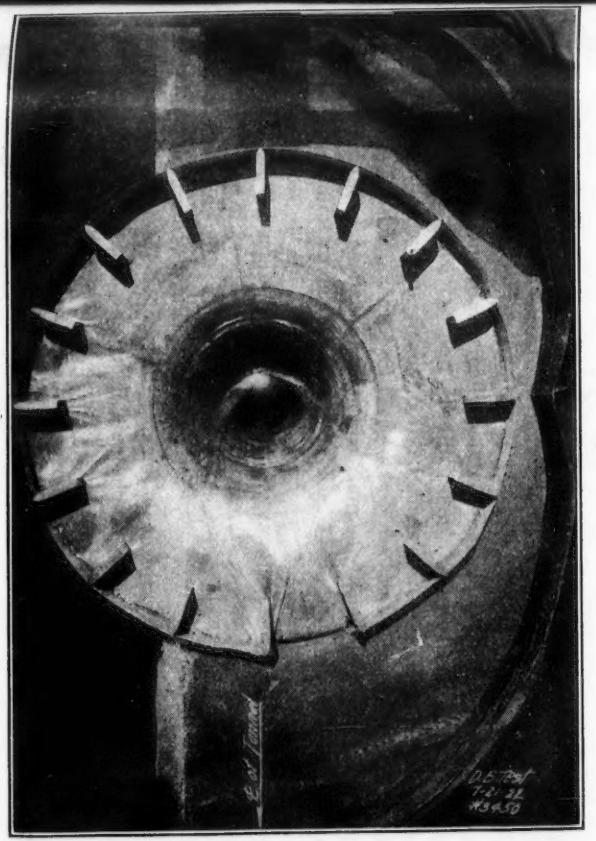


FIG. 23.—FLOW WITH FLASH-BOARDS IN PLACE, EXCEPT AT ONE SECTION BETWEEN PIERS.

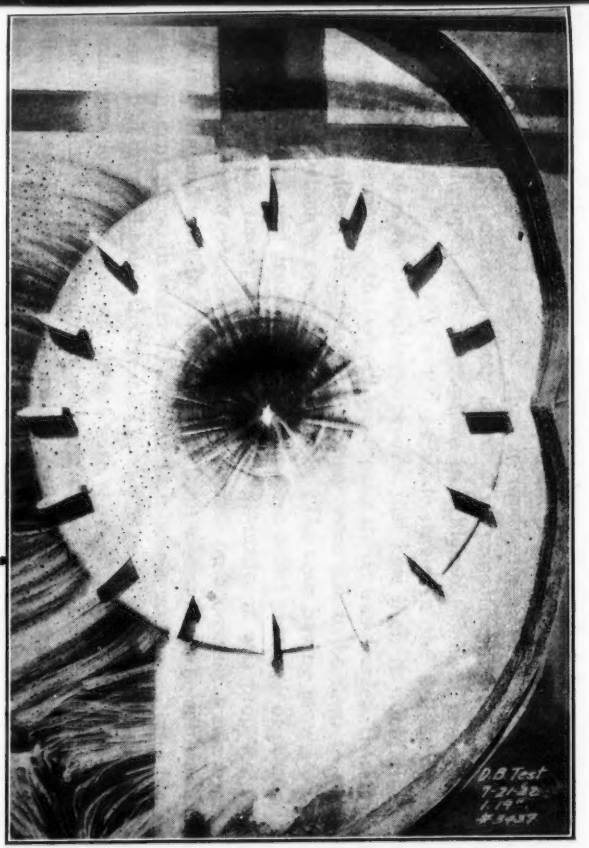
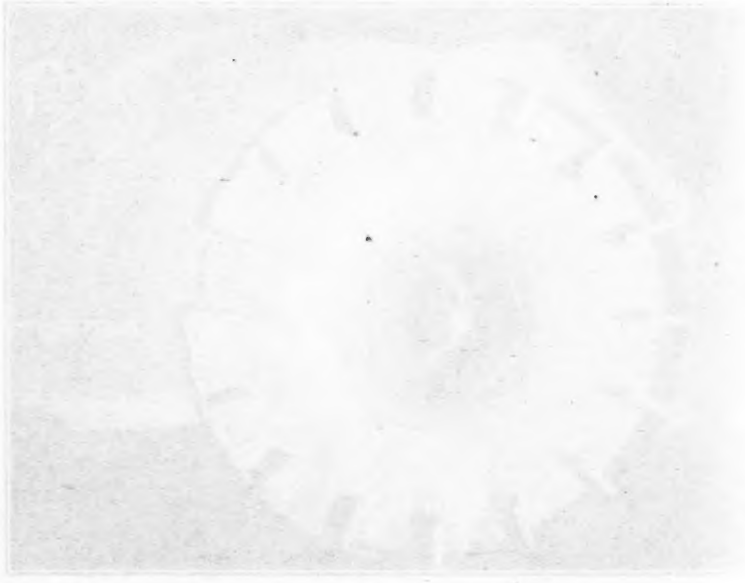


FIG. 24.—TYPICAL STREAM LINE PHOTOGRAPH WITH PIERS.



at the noses of the piers, must be very close to the truth. On comparing the photographs of the open-crest model with those of the flash-board type crest at the same discharges (see Figs. 9 and 21, and Figs. 19 and 22), it is noticeable that the piers have a beneficial effect in producing more uniform flow conditions around the entire spillway crest. The projecting nose of water which occurred on the open crest is nowhere in evidence. Such a projecting ridge of water (Figs. 9 and 19) would be undesirable in the full-sized spillway, especially as it is not possible to predict just how high it would rise or the amount of disturbance that would accompany it. It was decided, therefore, that the piers would be desirable even if they were not to be used to support a bridge from which to operate flash-boards. The tests having proven conclusive on this point, it was decided to adopt the flash-board pier type of crest.

At the time the Whitingham tests were made, it was proposed to use a traveling caisson or gate which would operate on a circular track around the spillway lip and serve to cut off the flow from a section of the spillway between two adjacent piers, in order to replace flash-boards on the spillway before the water had dropped below the crest. It was desirable, therefore, to determine the minimum length of pier consistent with a small quantity of back-water in a section where the flash-boards were being replaced while water was still flowing through the adjacent sections. This quantity of back-water was determined by trial, the pier models being cut off gradually until the length was only 4.8 in. With this length, there was a very small quantity of back-water at the line of flash-boards when the head on the spillway was as much as 2 or 3 in., corresponding to 6 or 9 ft. on the full-sized spillway. After the length of the piers had been determined, studies for the best shapes of these up-stream and down-stream ends were made by observing the contraction conditions at the piers and by marking the lines of contraction on the flat surface of the spillway model. For the down-stream ends of the piers, square ends, symmetrically pointed ends, and unsymmetrically pointed ends were tried, and it was found that square ends gave the best results. For the up-stream ends both symmetrically and unsymmetrically pointed shapes were tried, and the former proved to be superior. After all these details for the piers had been settled by observation, the capacity tests for the completed pier crest type were made giving the results which are shown on Fig. 17 and from which the capacity curve was made.

In order to determine the lateral pressures on a pier due to flow on one side only, a number of profiles of water surface were taken by marking the water-line on the pier with a lead pencil. These profiles were used later in the structural design of the piers.

The question arose as to whether or not a considerable depth of flow through only one or two openings between piers would produce conditions that might cause erosion of the concrete lining. To settle this, a test, as shown in Fig. 23, was made, indicating that the flow through even one opening tends to distribute itself quite uniformly around the entire cylindrical section of the shaft and, therefore, would not be injurious.

Complete sets of pressure readings were obtained, except in the case of Piezometer No. 5, for the flash-board pier type of crest. A comparison of the

resulting curves (Fig. 18), with the pressure curves, based on the Worcester tests of June 9 (Fig. 12), shows that the vacuum existing at various points in the model was, in general, less than that indicated by the Worcester tests and that of the two types of spillway the flash-board type is productive of smaller vacuums. Due to the fact that flow conditions in the Whitingham testing flume were so much better than those obtaining previously, the pressure readings also should be much more reliable.

From the pressure curves of Fig. 18, for the clear crest model with a spiral approach channel, the effective diameters of the stream at Piezometers Nos. 4 and 5 were computed, the details of the computations being shown in Table 9. For the design flow of 3.48 sec-ft., the diameters were 6.94 and 6.55 in., respectively, and for the maximum flow before flooding, of 4.30 sec-ft., the diameters were 7.58 and 7.28 in., respectively. It can be stated, therefore, that, in general, the spiral approach tends to reduce the partial vacuums in the spillway and also to reduce the magnitude of the Venturi tube effect.

TABLE 9.—STUDY OF PRESSURE AND VELOCITY CONDITIONS IN FREELY FALLING JET AND IN VERTICAL SHAFT AS DEVELOPED BY WHITINGHAM TESTS OF JULY 19, 1922, ON CLEAR CREST MODEL.

From the curves, for $Q = 3.48$ sec-ft. over the model, there is a vacuum of about 0.6 ft. at Piezometer No. 4 and 0.3 ft. at Piezometer No. 5. Therefore:			
Velocity head at Piezometer No. 4	$= 2.11 + 0.6 = 2.71$ ft.	Velocity	$= 13.24$ ft. per sec.
" " " " No. 5	$= 3.14 + 0.3 = 3.44$ ft.	"	$= 14.85$ ft. per sec.
Area of jet at Piezometer No. 4	$= \frac{3.48}{13.24} = 0.2627$ sq. ft.	Diameter	$= 0.5783$ ft. $= 6.94$ in.
" " " " " No. 5	$= \frac{3.48}{14.85} = 0.2342$ sq. ft.	"	$= 0.5455$ ft. $= 6.55$ in.
From the curves, for $Q = 4.30$ sec-ft. over the model, there is a vacuum of about 0.8 ft. at Piezometer No. 4 and 0.3 ft. at Piezometer No. 5. Therefore:			
Velocity head at Piezometer No. 4	$= 2.10 + 0.8 = 2.90$ ft.	Velocity	$= 13.65$ ft. per sec.
" " " " No. 5	$= 3.13 + 0.3 = 3.43$ ft.	"	$= 14.84$ ft. per sec.
Area of jet at Piezometer No. 4	$= \frac{4.30}{13.65} = 0.3150$ sq. ft.	Diameter	$= 0.6320$ ft. $= 7.58$ in.
" " " " " No. 5	$= \frac{4.30}{14.84} = 0.2896$ sq. ft.	"	$= 0.6065$ ft. $= 7.28$ in.

From the data of Tables 7 and 8 on the depth of flow at the discharge end of the tunnel model, the areas of the stream at that point were computed and the corresponding velocities obtained by dividing the areas into the observed flows. The flash-board pier type of crest having been adopted, with piers of the length and shape described, a capacity curve for the full-sized spillway was prepared from the proper curve on Fig. 17 and is shown on Fig. 25.

CONCLUSIONS

The writer believes that the foregoing theoretical and experimental investigations have shown that:

a.—Shaft spillways, if designed in accordance with correct hydraulic principles, can be relied on to act as predicted within the usual limits of precision of hydraulic designs involving the flow of water.

b.—The problem of the reduction in the capacity of shaft spillways due to the entraining of air is not a serious one if proper methods of design are followed.

c.—Conditions of partial or complete vacuum may exist in the converging section of shaft spillways, but they do not materially affect the capacity.

d.—If all or a section of the approach to such a spillway is to be through a comparatively narrow and shallow channel, flow conditions over the spillway will be greatly improved by the introduction of radial piers on the crest of the spillway.

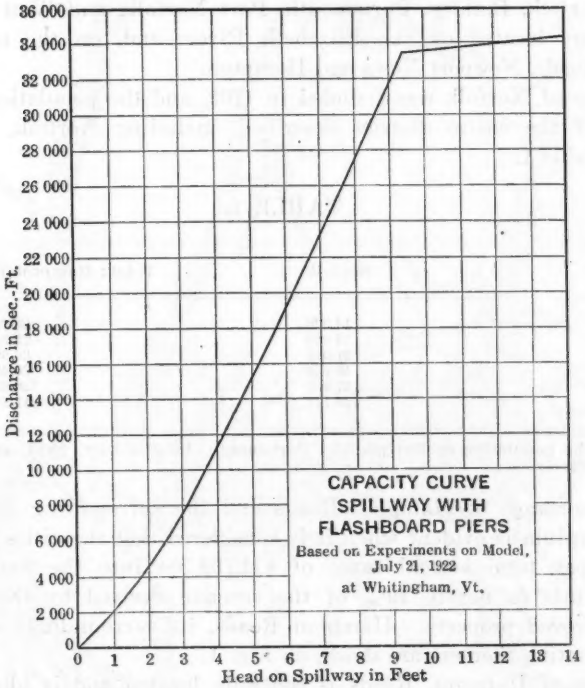


FIG. 25.

ACKNOWLEDGMENT

The work of design and testing was done by the writer for The J. G. White Engineering Corporation, with the assistance of W. P. Creager, M. Am. Soc. C. E., and under the general direction of A. S. Crane, M. Am. Soc. C. E. The field work of testing was carried out by A. C. Eaton, M. Am. Soc. C. E., for the Power Construction Company under the general direction of the writer.

THE RAILWAYS' PART IN THE DEVELOPMENT OF HAMPTON ROADS*

By Charles S. Churchill,† M. Am. Soc. C. E.

Hampton Roads is formed by the estuaries of the James, Nansemond, and Elizabeth Rivers on the west and south, and is limited on the east by Old Point Comfort. The principal industrial and residential sections surrounding Hampton Roads are in their order, beginning on the southeast side of Hampton Roads: Norfolk, Berkley, Portsmouth, Port Norfolk, and West Norfolk—all of which are located on the Elizabeth River—and, on the north side of Hampton Roads, Newport News and Hampton.

The City of Norfolk was founded in 1705, and the population of Norfolk and that of the entire district described, including Norfolk, increased as shown in Table 1.

TABLE 1.

Year.	Norfolk.	Total: Hampton Roads District.
1850	14 326	23 235
1860	14 620	25 964
1880	21 966	36 988
1900	46 624	86 450
1910	67 452	126 352
1920	115 777	211 898

Note.—The population of Norfolk and Portsmouth, Va., in July, 1923, was 159 086 and 57 341, respectively.

The importance of Hampton Roads and the surrounding district to the State of Virginia is evident when it is considered that the cities and counties around it pay total annual taxes of $11 702 704 into the State Treasury, which amounts to nearly 19% of the revenue derived by the State from taxable improved property. Hampton Roads, its surroundings and the railroads terminating thereon, are shown on Fig. 1.

The Port of Hampton Roads is centrally located and is ideally situated from the standpoint of climate, accessibility to the interior, and proximity to old established world trade routes.

Although the depth of the entrance to the Harbor of Norfolk and Portsmouth was originally 21 ft., it was deepened to 25 ft. in 1876, to 28 ft. in 1898, and to 30 ft. in 1907. In 1911, there was completed a channel 30 ft. deep, with a width of 600 ft. from Hampton Roads to Lamberts Point, and from Lamberts Point to the junction of the Southern and Eastern Branches of the Elizabeth River 30 ft. deep and 800 ft. wide. In 1912, the channel was 35 ft. deep and 400 ft. wide from Hampton Roads to The Belt Line Bridge over the Southern

* Presented at the Fall Meeting of the Society, at Richmond, Va., on October 18, 1923.

† Vice-Pres., in Chg. of Purchases, Real Estate and Valuation, Norfolk & Western Ry., Roanoke, Va.

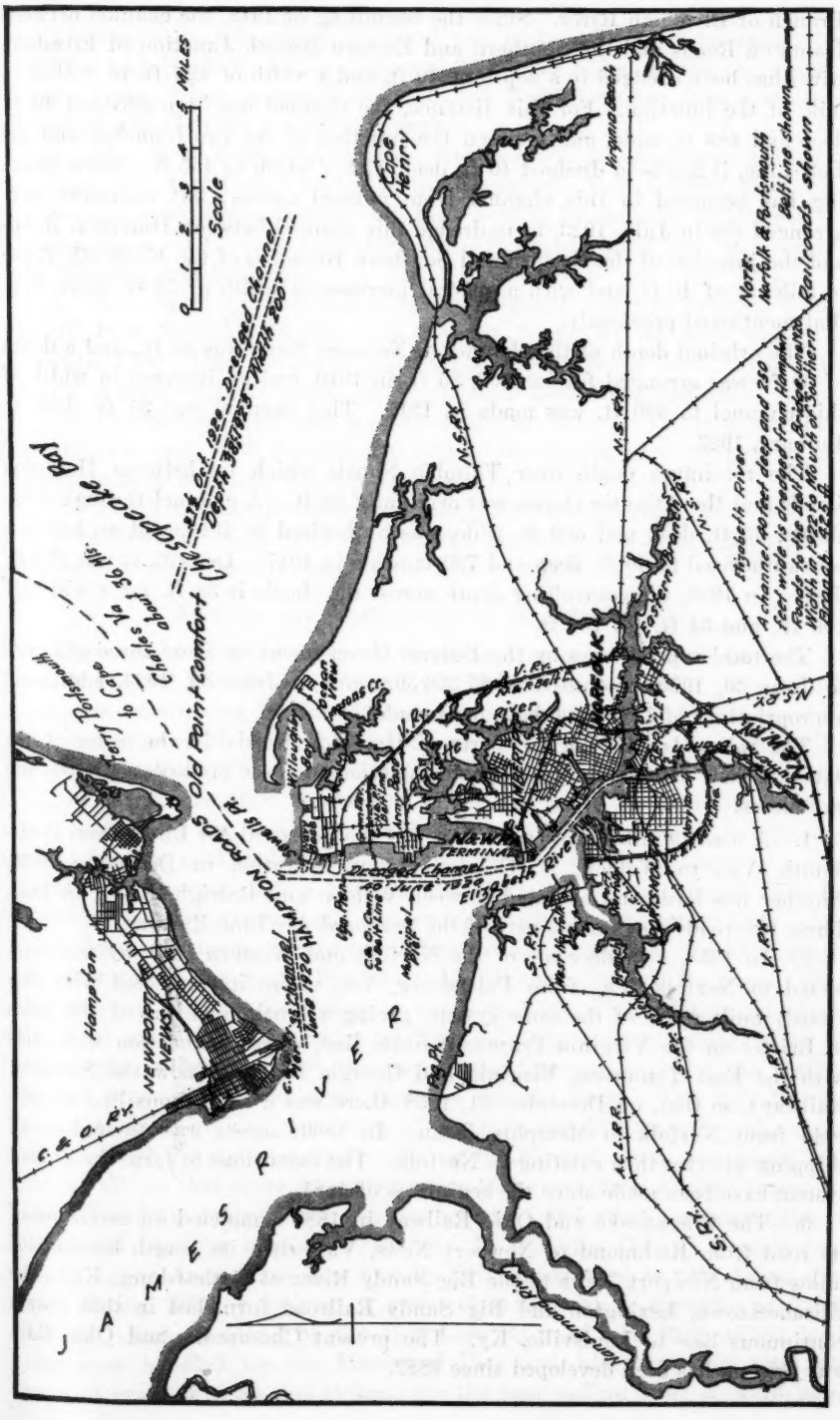

Scale
Cape Henry
Virginia Beach
Norfolk & Portsmouth Belt Line shown
Railroads shown
Elizabeth River
Old Point Comfort
Chesapeake Bay
Hampton Roads
Newport News
Hampton
Norfolk
Portsmouth
Elizabeth River
Deepened Channel
Eastern Branch
Western Branch
James River
N. & W. Ry.
V. Ry.
S. Ry.
A.C.L. R.R.
C. & O. Ry.
N.S. Ry.
Lambert's Point
Sewall's Pt.

FIG. 1.

Branch of Elizabeth River. Since the beginning of 1918, the channel between Hampton Roads and the Southern and Eastern Branch Junction of Elizabeth River has been dredged to a depth of 40 ft. and a width of 425 ft. to within $\frac{6}{10}$ mile of the junction. For this distance, the channel has been dredged 40 ft. deep and 485 ft. wide, and between the junction of the two branches and the Belt Line, it has been dredged 40 ft. deep with a width of 450 ft. Some shoaling has occurred in this channel from natural causes; but contracts were arranged for in July, 1923, to re-dredge this channel between Hampton Roads and the junction of the Eastern and Southern Branches of the Elizabeth River to a depth of 40 ft. and with a general increase in width of 50 ft. more than that mentioned previously.

The original depth of the channel to Newport News was 25 ft., and a depth of 30 ft. was arranged for in 1902, 35 ft. in 1910, and an increase in width of this channel to 600 ft. was made in 1917. This channel was 35 ft. deep in January, 1923.

The minimum depth over Thimble Shoals which lie between Hampton Roads and the Atlantic Ocean was originally 30 ft. A channel through these shoals, 35 ft. deep and 500 ft. wide, was authorized in 1910, and an increase was authorized to 40 ft. deep and 750 ft. wide in 1917. In 1920, it was 37½ ft. deep. In 1923, the controlling depth across the shoals is 38 ft. for a width of 200 ft., and 34 ft. for 750 ft.

The total expenditures by the Federal Government on these three channels to June 30, 1920, amounted to $7 336 592, and to June 30, 1923, additional appropriations of $1 555 000 have been made.

The railroads entering the ports of Hampton Roads, in the order of the dates of their completion, or of the completion of their predecessor roads, are as follows:

1.—A road 79 miles long (predecessor of the Seaboard Air Line) from Portsmouth, Va., to Weldon, N. C., was put in operation in December, 1836. Another was built and operated between Weldon and Raleigh, N. C., in 1853. These two roads now form a part of the Seaboard Air Line Railway.

2.—In 1858, a predecessor of the Norfolk and Western Railway was completed to Norfolk, Va., from Petersburg, Va., where it connected with previously built roads of the same system, giving a continuous line of 408 miles to Bristol on the Virginia-Tennessee State line, where connection was made with the East Tennessee, Virginia and Georgia Railway (now the Southern Railway), so that, on December 31, 1858, there was a continuous line of railroad from Norfolk to Memphis, Tenn. In 1860, access was secured to all shipping wharves then existing in Norfolk. The extensions to form the present system have been made since the beginning of 1881.

3.—The Chesapeake and Ohio Railway in 1882 completed an extension of its road from Richmond to Newport News, Va., when its length became 405 miles from Newport News to the Big Sandy River at Catlettsburg, Ky. The Elizabethtown, Lexington and Big Sandy Railroad furnished in that year a continuous line to Louisville, Ky. The present Chesapeake and Ohio Railway System has been developed since 1882.

4.—The Norfolk-Southern Railroad was built and put in operation from Berkley and Norfolk, Va., to Edenton, N. C., 73 miles, at the beginning of 1883. Later, it was extended south through North Carolina, also westward to Charlotte, N. C., with numerous branches.

5.—The New York, Philadelphia and Norfolk Railroad was built from Delmar, Del., into Cape Charles on the Chesapeake Bay, in 1884, from which connection is secured to Norfolk and Port Norfolk by steamers and barges, thus furnishing a direct route to Philadelphia, Pa., and points west and north on the Pennsylvania Railroad.

6.—The Atlantic Coast Line (predecessor road) was built between Pinners Point, Port Norfolk, 100 miles, to Tarboro, N. C., on the main line of the Atlantic Coast Line Railroad, and opened for traffic in April, 1890, giving access to other southern points on the Atlantic Coast Line Railroad.

7.—The Southern Railway's predecessor line between Danville and Portsmouth, Va., and West Norfolk, 205 miles, was opened in the spring of 1890, which route gave access to other points on the Southern Railway.

8.—The Virginian Railway was opened for through traffic on April, 1909, between Sewalls Point, Norfolk, Va., and Deepwater, W. Va. (junction with Chesapeake and Ohio Railway), a distance of 444 miles.

Norfolk and Portsmouth Belt Line Railroad

These eight lines of railroads entering Hampton Roads have been connected by the Norfolk and Portsmouth Belt Line Railroad. Prior to 1898, the lines terminating in Norfolk had added float-bridges, and cars were interchanged between the several railroads, and handled by barges across the Bay, by the New York, Philadelphia, and Norfolk Railroad in both directions from Cape Charles. In 1896, the late A. J. Cassatt, while President of that road, proposed a plan for having one yard at Port Norfolk where all freight designated for that road could be accumulated and transferred to barges at a single point. To accomplish this purpose, the Norfolk and Portsmouth Belt Line Railroad was constructed and put in operation in September, 1898. Additions have been made thereto from time to time, including one for industrial development on the South Branch of the Elizabeth River. Its present extent is shown on Fig. 1: Beginning at the New York, Philadelphia and Norfolk Railroad yard and float-bridge at Port Norfolk and extending to another float-bridge constructed for the use of the Chesapeake and Ohio Railway, located between the Municipal Pier and Sewalls Point. This belt line thus forms a circle around the Cities of Norfolk and Portsmouth and connects with each of the eight railroads entering the Norfolk ports, with equal facilities to all, so that since its construction there has been a great increase in industrial development along the route thereof and great savings in the cost of freight handling have been secured. In 1921, the City of Norfolk leased a part of the Army Base near Sewalls Point designated as "Municipal Pier", and, during 1922, completed a grain elevator with a capacity of 750 000 bushels, which is served by the Belt Line Railroad. In 1922, 14 361 loaded cars were handled for the Municipal Pier and Elevator. The total shipment of grain through this elevator for the year ending June 30, 1923, aggre-

gated 1 729 000 bushels. The Municipal Terminals are so connected by the Norfolk and Portsmouth Belt Line Railroad that they are really union terminals, in that arrangements are in effect whereby all freight, when properly routed, coming into Norfolk for export or in moving out from these terminals may be handled on one bill-of-lading, the switching charge by the Belt Line being absorbed in the Norfolk rate.

The Belt Line also serves more than 100 other industrial developments. This line represents the joint investment of all the railroads that connect at Hampton Roads. The Norfolk and Portsmouth Belt Line Railroad is one of the most effective contributions of the eight railroads entering Norfolk and vicinity for the development of Hampton Roads. Proof of the value of its facilities, combined with those of the terminals of the eight railroads, is indicated by the fact that during the World War it was probably the only Eastern port free from congestion, through which great quantities of materials and large numbers of men were transported to and from Europe.

Table 2 shows for the years stated the total number of cars handled from one line to another, from industries to connections, and from connections to industries on the Belt Line.

TABLE 2.—Norfolk and Portsmouth Belt Line Railroad: Statement of Loaded and Empty Cars Handled During the Following Years.

Year.	Loaded.	Empty.	Total.
1912	136 762	77 726	214 488
1915	149 848	105 273	255 121
1916	180 178	100 489	280 667
1917	197 436	100 300	297 736
1918	216 375	54 292	270 667
1919	200 279	63 345	263 624
1920	190 141	65 357	255 498
1922	192 543	104 808	297 351

The previous statement includes the following loaded cars moved by the Belt Line to and from the U. S. Army Base:

Years.	Cars.
1917	256
1918	18 557
1919	23 021
1920	15 665
1922	3 411

Seaboard Air Line Railway

The Hampton Roads Terminal of the Seaboard Air Line Railway is Portsmouth, Va., and there is connected with that terminal about 6.3 miles of tracks and six warehouses. The tonnage available from the Seaboard Air Line for transshipment through Hampton Roads is that which originates in the Southern Border States, and consists of cotton, naval stores, fertilizing materials, and lumber. The routing of these materials is controlled largely by frequency of sailings, ocean rates, and inland rates to a port.

These conditions naturally divide the tonnage between ports available to the Seaboard Air Line Railway, of which Hampton Roads is the one at the northern end of the line.

The total tonnage received from and delivered to water carriers on Hampton Roads, by years, was as follows:

Years.	Tonnage.
1912	280 805
1917	282 366
1918 ...(Record not kept during Federal Control)	
1920	143 262
1922	155 922

The total of this tonnage for the years stated, that was handled by water carriers, is about 16% of the entire freight received and forwarded by the Seaboard at Portsmouth.

Norfolk and Western Railway

The records of the Norfolk and Western Railway and its predecessor companies show that the year the line was opened—1858—6 174 bales of cotton were received at Norfolk for water transshipment. In 1874, this had increased to 467 561 bales. The First Annual Report of the Norfolk and Western Railroad, in 1881, states that the road transported 325 862 bales of cotton, and during that year the total handled through Norfolk was 713 036 bales, but the statement is made that the business through Norfolk was then limited by lack of vessels en route for England. In 1881, the Norfolk and Western Railroad constructed its own wharves on the East Branch of the Elizabeth River at its Norfolk freight yard, and extensive warehouses were also constructed for the transfer of freight between cars and vessels. In 1886, in addition to these warehouses, the Company built a grain elevator with a capacity of 150 000 bushels. In 1882, it commenced the construction of a terminal line through Norfolk to Lamberts Point, preparing for transshipment of Pocahontas coal, the development of which had begun in 1881 and for shipment of which a line was also being built by the Norfolk and Western from Radford, Va., to Pocahontas, Va. The first coal pier was completed at Lamberts Point in the spring of 1885 and, during that year, shipments by vessels of Pocahontas coal began to New England, the Southern States, California, the West Indies, and South America. The depth of water at this pier, then accommodating the largest vessels, was 26 ft. at low tide. Following the year 1885, large extensions through West Virginia were made, in connection with the development of the coal territory of the Company. Iron Coal Pier No. 2 was constructed in 1889, Steel Pier No. 3, still larger, was built in 1901, and Coal Pier No. 4 (Figs. 2 and 3), which is 90 ft. high and provided with electrically operated elevators and car dumpers, was built during 1912 and put in operation in 1913, its capacity being 6 600 tons per hour. Prior to 1890, two large warehouses had also been constructed at Lamberts Point for the handling of miscellaneous freight to and from steamers. The

improvement in the rapid operation in handling coal by the Norfolk and Western coal piers is indicated by the fact that coal handled over the last pier built, which is electrically operated throughout, was, during 1921 and 1922, at five times the average speed developed on Pier 3 in 1912.

In common with all roads having termini on or near the mouth of Elizabeth River, a large amount of dredging has been necessary, not only between the Port Warden Line and the channel of the river for entrance, but also for anchorage. Heavy expenditures were made therefor by the Norfolk and Western Railway. The depth of water maintained at Lamberts Point Terminals is 35 ft. at low tide. The Lamberts Point Terminal property has a total length of 8 533 ft. of water-front and the yards connected therewith contain 108 miles of track. Since 1915, the piers and warehouses for freight have been increased to a total of nine piers, on four of which are warehouses. All three of the existing coal piers are now equipped for complete electrical operation, having a capacity of about 100 000 tons per day and a working capacity under general conditions of 60 000 tons per day. The total expenditure for terminal facilities at Lamberts Point aggregates about $7 400 000.

The net tons of freight delivered to and received from water carriers on Hampton Roads, by years, was as follows:

Years.	Tonnage.
1912	615 371
1917	1 183 653
1918	1 187 131
1920	315 287
1922	229 193

This is exclusive of the miscellaneous freight handled in part by the Norfolk and Western Railway and thence by the Norfolk and Portsmouth Belt Line to the Army Base prior to its conversion into a Municipal Pier, also exclusive of traffic that moved over the Norfolk and Western Railway to the junction of the Virginian Railway at Norfolk and was switched by the latter to the Naval Base. The traffic through the Army and Navy Bases is included as carloads by years under the headings "The Belt Line Railroad" and "The Virginian Railway."

The shipments of coal over the Lamberts Point piers of the Norfolk and Western Railway is given in Table 3 under "Coal Shipments Through Hampton Roads." The Norfolk and Western Railway secures through the Municipal Pier regular steamship service to the principal foreign ports; also through its terminals.

The termini of owned tracks of the Norfolk and Western Railway are Durham, N. C., Winston-Salem, N. C., Hagerstown, Md., Bristol, Va.-Tenn., Norton, Va., Cincinnati, Ohio, and Columbus, Ohio, where proper arrangements have been made for receiving freight from the South and West, as well as from the Northwest, for shipment to Hampton Roads, or for handling such shipments received through Hampton Roads primarily to Chicago, Ill., St. Louis, Mo., Memphis, Tenn., New Orleans, La., etc. Because of

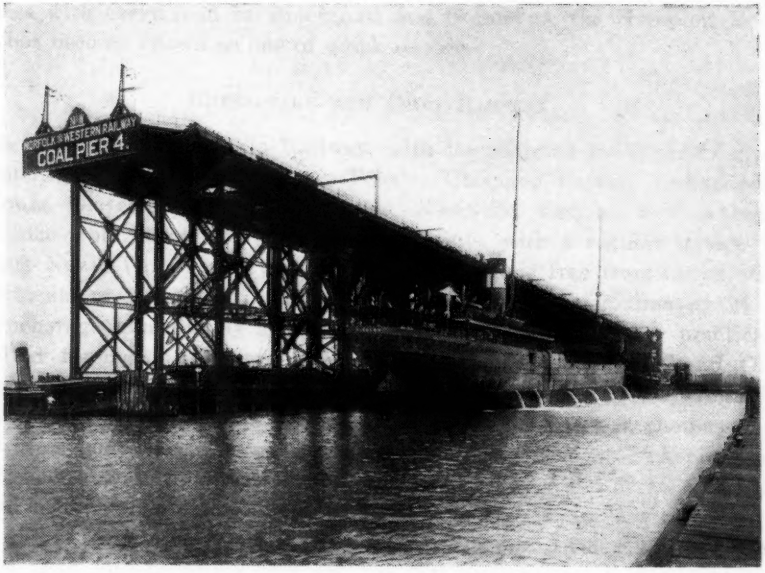


FIG. 2.—VIEW OF COAL PIER NO. 4, NORFOLK AND WESTERN RAILWAY.

FIG. 3.—VIEW SHOWING CAR DUMPERS, COAL PIER NO. 4, NORFOLK AND WESTERN RAILWAY.

its complete terminal facilities at all points, particularly its adequate transfer facilities with every road in Cincinnati and Columbus, the Hampton Roads route has become known as one of quick service.

Chesapeake and Ohio Railway

The Chesapeake and Ohio Railway, with its affiliated lines, serves direct Cincinnati, Louisville, Columbus, Toledo, Chicago, Peoria, Indianapolis, St. Louis, Cleveland, Buffalo, Memphis, Nashville, etc., as well as handling traffic from beyond or through these points, with a regular service *via* Newport News, Va. The harbor at Newport News is free from ice or other impediments to navigation, and has a tidal range of approximately 2½ ft. The anchorage immediately adjacent to the piers is available for practically unlimited tonnage. The terminal facilities of the Chesapeake and Ohio Railway, at Newport News, are sufficient for handling a large volume of general traffic, as well as coal. They include 100 miles of track, one passenger and express pier, 402 ft. long and 600 ft. wide; two open or deck piers, one 650 ft. long and 65 ft. wide, and the other, 790 ft. long and 52 ft. wide; five covered merchandise piers ranging from 457 to 810 ft. in length and from 56 to 194 ft. in width; cattle pens with a capacity of 1 200 head; two float-bridges from and to which cars are handled by car-floats to points within Hampton Roads Harbor; and two coal piers, one of steel and concrete, electrically operated, 1 200 ft. long, 75 ft. wide, and 90 ft. high, with a capacity of 6 600 tons per hour and a working capacity under general conditions of 36 000 tons per day. The other pier is of wooden construction, incline operation, 800 ft. long, 55 ft. wide and 60 ft. high. The first coal pier was completed in the early part of 1882, but coal for fuel purposes was delivered to 106 ocean steamers in the latter part of 1881.

In 1884, the Chesapeake and Ohio Railway built at this point Grain Elevator A, with a capacity of 1 775 000 bushels; and, in 1900, Elevator B, with a capacity of 1 000 000 bushels was constructed. The first elevator was destroyed by fire in 1915, and all business is now handled by Elevator B. The following is a statement, by years, of grain, in bushels, handled through the elevator, beginning with 1913:

Years.	Bushels.	Years.	Bushels.
1913	10 013 525	1918	19 959 888
1914	2 647 997	1919	7 680 972
1915	47 896 016	1920	2 376 881
1916	52 444 111	1921	494 378
1917	32 344 430	1922	713 362

Newport News has regular steamship service to and from the principal United Kingdom and Continental ports, as well as coastwise service. The adaptability of Newport News as a port of export was recognized by the Government during the Spanish-American War, as well as during the World War, and during the latter period there was handled through this port 115 083 soldiers, 350 000 horses and mules, and 4 134 000 tons of general cargo and

war supplies, in addition to grain exported in accordance with statement previously given, and in addition to its shipments of coal included in Table 3.

Norfolk-Southern Railroad

The Hampton Roads Terminal of the Norfolk-Southern Railroad is at Berkley at the junction of the Southern and Eastern Branches of the Elizabeth River, and its terminal yard contains about 5½ miles of track.

The total tonnage received from and delivered to water carriers, by years, was as follows:

Years.	Tonnage.
1917	431 018
1918	233 202
1920	233 761
1921	352 023
1922	442 112

New York, Philadelphia, and Norfolk Railroad

The operation of this road was started in November, 1884, in the latter half of which month regular connection from Cape Charles by boat to Norfolk was begun, the first cargo barge having commenced operation between Cape Charles and Norfolk in March, 1885.

The number of loaded cars handled from Cape Charles across the Bay and through Hampton Roads to and from Norfolk, by years, were as follows:

Years.	Loaded Cars.
1912	185 407
1917	247 668
1918	203 332
1920	196 321
1922	235 823

The New York, Philadelphia and Norfolk Railroad has secured property for a terminal to be located southeast of Ocean View, at the mouth of Little River, where a float-bridge will ultimately be constructed with proper terminal tracks. Right of way has also been secured so that track will ultimately be built to a connection with the Norfolk and Portsmouth Belt Line. Under this plan, the distance from Cape Charles to the Norfolk and Portsmouth Belt Line will be shortened, with considerable saving in time.

Atlantic Coast Line Railroad

Hampton Roads forms the northernmost tide-water port of the Atlantic Coast Line Railroad, its terminal facilities being located at Pinners Point, Portsmouth, where four warehouses and 23 miles of track connected therewith are maintained.

The total tonnage received from and delivered to water carriers on Hampton Roads, by years, was as follows:

Years.	Tonnage.
1912	215 888
1917	173 947
1918	...(Record not kept during Federal Control)
1920	104 427
1922	120 448

Southern Railway System

The Hampton Roads Terminal of the Southern Railway is located at Pinners Point, this terminal being provided with four warehouses and 27 miles of track. The Southern Railway began operating its trains in and out of this terminal in January, 1896. Prior to that date, its water-borne traffic was handled through West Point. The Atlantic and Danville Railway, the predecessor company of the Southern Railway, was opened for operation from West Norfolk to Danville, Va., in 1890. Under an agreement executed in August, 1895, between the Southern Railway Company and the Washington and Weldon Railroad Company, and one with the Norfolk and North Carolina Railroad Company under the same date, the Southern Railway acquired the use of a line between Selma, N. C., and Pinners Point, Va. It was at this time that the Southern Railway completed the development of its piers, warehouses, etc., at Pinners Point, and established its through route from the West known as the "Asheville Line", whereby business was carried to Pinners Point Terminal *via* Asheville, Salisbury, Greensboro, Selma, and Tarboro, N. C. With the establishment of the Pinners Point Terminal, in 1896, the business formerly handled through the Atlantic and Danville Terminal, at Portsmouth and West Norfolk, was concentrated as far as practicable through the Pinners Point Terminal. In the latter part of 1917, the United States Government took charge of the Southern Railway's export pier at Pinners Point, and this accounts for the light tonnage reported for 1918. During the latter part of 1918, the Clyde Line abandoned its service between Norfolk and Philadelphia, and, in 1920, the Old Dominion Steamship Company discontinued service between Norfolk and New York. These were the Southern's heaviest connections. Although other lines have since entered the field, the business has never approximated its former proportions.

The movement of traffic through the Port of Hampton Roads by the Southern Railway was as follows:

Years.	Tonnage.
1912	562 581
1917	445 850
1918	282 970
1920	330 491
1922	378 246

The Virginian Railway

The Hampton Roads Terminal of the Virginian Railway is located at Sewalls Point, which has recently become a part of Norfolk. This terminal

is equipped with 31.9 miles of track and a large electrically operated steel coal pier, having a capacity of about 50 000 tons per day and a working capacity under general conditions of 36 000 tons daily. Aside from its coal terminal at Sewalls Point, the Virginian Railway was without water-front facilities at Hampton Roads until the opening of the Army Supply Base, except that arrangements were made first with the Southern Railway at Pinners Point and afterward with the Norfolk and Western Railway at Lamberts Point for the handling of the slight export business originating on the Virginian Railway. This traffic, however, on account of abnormal trans-Atlantic transportation conditions caused by the war, was negligible in amount from 1912 to 1922. In 1912, the Virginian Railway handled 19 161 tons of export freight to Norfolk. This business began to revive during the latter part of 1922, but has not yet reached the proportions of 1912.

The Naval Operating Base at Sewalls Point, connected by tracks exclusively with the Virginian Railway, was the scene of much activity during the months that the United States participated in the World War. All carload freight received and forwarded from the Naval Operating Base was handled in switch movement by the Virginian Railway. The switch movement by years was as follows:

Years.	Cars.
1918	15 822
1919	13 662
1920	5 790
1922	3 473

During the period when the railroads were under Federal Control, the Virginian Railway handled into and out of Norfolk very little miscellaneous traffic, other than business originating on or destined to points local to the road. The Deepwater, W. Va., Gateway (connection with Chesapeake and Ohio Railway) was closed in conformity with decision of Government transportation experts; and the Virginian Railway gave its entire energy to the movement of bituminous coal to tide-water. The shipment of coal over its Sewalls Point Pier is given in Table 3.

Coal Shipments Through Hampton Roads

The Norfolk and Western Railway ships over its Lamberts Point Piers Pocahontas coals as well as high volatile coals from its road in Virginia and West Virginia. The Chesapeake and Ohio Railway ships coals from the New River and Kanawha Districts over its piers at Newport News. The Virginian Railway handles Pocahontas and New River measures of coal from its territory in West Virginia over its Sewalls Point Pier. All these coal-fields are within a distance of about 400 miles from the terminal piers, and the facilities furnished by the three roads are such that the time in transit is relatively short. Table 3 shows, by years (1912 to 1922, inclusive), the gross tons of coal dumped at these three terminals.

TABLE 3.

Year.	Norfolk & Western, Lambert's Point, gross tons.	Chesapeake & Ohio, Newport News, gross tons.	Virginian, Sewalls Point, gross tons.	Hampton Roads, Total gross tons.
1912	5 213 766	3 762 878	2 474 052	11 450 696
1913	5 598 716	3 112 015	3 283 873	11 994 604
1914	5 987 655	3 231 355	2 847 335	12 066 345
1915	7 530 604	4 162 850	3 119 021	14 812 475
1916	7 450 802	4 828 254	4 287 566	16 566 622
1917	6 321 457	4 892 869	4 429 296	15 643 622
1918	7 914 648	5 011 245	4 220 778	17 146 671
1919	5 874 510	3 436 364	3 618 384	12 929 288
1920	8 807 803	7 264 390	5 497 131	21 569 324
1921	6 820 341	4 321 906	4 198 657	15 340 904
1922	6 929 882	3 247 193	4 732 535	14 909 610

Published reports show that, in 1919, Hampton Roads was the third Atlantic Coast port in amount of water-borne tonnage handled, and that for the year ending June 30, 1921, it was second in total export and import tonnage.

The tonnage through Hampton Roads for the calendar year 1922, in net tons, was made up as follows:

Exports and imports of coal	1 193 921	tons
" " " " miscellaneous freight	948 245	"
Total exports and imports	2 142 166	"
Total coastwise coal	15 504 842	"
Coastwise miscellaneous freight, at least	452 992	"
Total tonnage through Hampton Roads	18 100 000	"

exclusive of 235 823 loaded cars transferred through Hampton Roads by the New York, Philadelphia and Norfolk Railroad.

In the foregoing, a general description is given of the facilities furnished and the character of the improvements made by the railroads that enter any of the terminal ports of Hampton Roads. For example, one item not mentioned in detail is the fact that coal piers of the Virginian, Chesapeake and Ohio, and the Norfolk and Western Railways are capable of unloading and handling coal cars having a carrying capacity of 100 to 120 tons. In other words, railroad facilities have been kept strictly up to the requirements of the day. The map of Hampton Roads (Fig. 1) locates some of the controlling channels, giving depths thereof, a description of which is given in some detail in the first part of the paper. The fact that from time to time at least 33 vessels—having a draft of 32 ft. or over when loaded—enter Hampton Roads is evidence of the need of some expedition in dredging, not only of that already contracted for, but also some additional work of the same character, including also the dredging of anchorage space in the Harbor of Norfolk, with moorings provided, so that vessels may be located within easy call from the piers at which they are to take cargoes.

Under an Act of the General Assembly of the State of Virginia, March 16, 1920, a Commission for the development of Hampton Roads was appointed

by the Governor. This Commission was duly organized February 2, 1921, when N. D. Maher, President of the Norfolk and Western Railway, was elected Chairman, and H. L. Ferguson, President of the Newport News Shipbuilding and Dry Dock Company, was chosen Vice-President. This Commission submitted its first report, being House Docket No. 5, under date of January 3, 1922. It recommended the establishment of a permanent State Commission of six members which is now working on the formation of plans, such as for dredging for improvement of the harbor, hard surface roads to all harbor cities, and unification of railroad and steamship terminals.

THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI*

A SYMPOSIUM

PAGE

The Revival of Commercial Transportation on the Mississippi.
By M. J. Sanders, Esq. 2022

The Economics of Transportation on the Mississippi River.
By Joseph E. Ransdell, Esq. 2028

With Discussion by Messrs. Lewis Muhlenberg Haupt, C. E. Grunsky, and William T. Lyle.

* Presented at the Spring Meeting of the Society at New Orleans, La., on April 18 and 19, 1923, and continued from November, 1923. *Proceedings.*

THE REVIVAL OF COMMERCIAL TRANSPORTATION ON THE MISSISSIPPI

By M. J. Sanders,* Esq.

As the pioneers pushed westward in developing the country, the great Mississippi River and its tributaries were of vital importance as the only means of transportation. The fact that the navigability of these rivers in their natural state depended entirely on climatic conditions, and that for weeks or even months there was insufficient water over the shoals, was of course inconvenient, but was accepted, like the weather, as beyond control—and inevitable.

As the country was developed and the density of population increased, the enterprise of the railroad pioneers furnished more expensive but far more convenient transport, and, as was logical, the uncertain river channels were abandoned gradually, except for the shipping of the cheaper and bulkier commodities, for which the cost of carriage was a serious factor but time was of less importance.

Permanent navigable channels are essential for efficient and economic transportation, but prior to the Civil War little consecutive effort was made by the Federal Government to this end on the great river and its tributaries. For many years thereafter, there was great wastage of the public funds, not so much because the engineers failed to realize the importance of the problems and the magnitude of the work, but by reason of political difficulties which prevented the consideration and adoption by the Government of a comprehensive project along definite lines, backed by the necessary funds.

In their arguments before Congress for improved navigable channels, those who lived on the banks of the river, knowing the necessity of the cheaper water transportation as well as its advantages, were confronted by the indifference of the people living remote from the rivers, who objected to furnishing their share of the cost of improvements, without any return. They had also to face the antagonism of railroad executives, who, with some apparent justice, held it to be unfair for the Government to furnish a free highway on the rivers such as would deprive them of traffic which their facilities could handle readily, and for which in good faith they had expended liberally the funds of their stockholders.

These factors in opposition have retarded seriously all work for the improvement of rivers, and notwithstanding the years of earnest labor by waterway advocates, the hundreds of commercial associations, and the countless conferences and conventions, Congress has not yet adopted any really comprehensive and efficient plan of river improvement by which the commerce of the country might be fully served.

Meanwhile, commerce has developed so rapidly that it has far exceeded railroad capacity which for a long period was in excess of the requirements,

* Mgr., International Mercantile Marine and the Frederick Leyland Company, Limited, New Orleans, La.

and which helped many of the roads into bankruptcy. To-day, railroad officials themselves are voicing their alarm at the prospect. The following is quoted from a recent statement of a leading railroad executive:

"Traffic on our American railroads, measured in ton-miles, doubles about once in a decade. The next time our country has a real revival in business, we shall in all probability be confronted with the most severe congestion of railroad traffic and the greatest inadequacy of railroad facilities ever experienced in our history. When that happens, rates will be lost sight of. Every one will be clamoring for service. * * * Then business men will not be bothering themselves much about rates. All they will be thinking about will be how to get transportation at any price."

Instead of increasing facilities in proportion to the volume of commerce, there has been an actual decrease in rail mileage in recent years, and nothing approaching the needed increase in rolling equipment and terminals.

In the past ten years, it has been increasingly difficult to obtain the needed funds for proper railroad development, and although doubtless this has been influenced greatly by conditions beyond the control of railroad executives, it is serious, and, at present, there is no assurance that the enormous additional capital requisite can be obtained. In any case, unless in fact railroad transportation is the most economical and most efficient by which to meet the necessities of domestic transportation, it is surely the height of folly to ignore other transportation methods, as has been done in the past. Economy and efficiency are as essential in transportation as in any other industry. Even with the best known methods, it is apparent that the cost of rail transportation cannot be reduced appreciably and is more likely to be increased. It is necessary, therefore, to give serious consideration to other methods.

The motor truck is becoming a factor of great value for short-haul traffic. Good highways are adding materially to transportation facilities, although the cost, especially that of upkeep, does not indicate great economy, unless under special conditions.

The transportation possibilities of those waterways that already have been improved for several thousand miles, and are capable of enormous addition, will now, perhaps, be regarded seriously. One important factor of water transportation is its relative cheapness. It is difficult, of course, to place an arbitrary figure of cost on any form of transportation because so many features affect it: The density of the traffic; the physical conditions; the nature of the traffic; the important question of terminals, their cost and value; as well as the character of the transporting factor. To be of value, comparisons must cover a variety of conditions throughout an extended period.

It is beyond serious controversy, however, that, under proper physical conditions, and with a reasonable average traffic density, the ton-mile cost of carrying freight on a navigable waterway is much less than is possible by any rail or highway transport. Beyond the fact that rivers are public highways, improved and maintained as a general responsibility of the Federal Government, and that thereby transportation is relieved of the enormous and inevitable burden of tracks or highways, the physical conditions dictate the great economy of haulage by water. Further, the cost of floating equipment per

ton-mile of water transport is much less than that necessary for the rolling equipment of railroads or highways. So influential are these factors, that the relative flexibility of the motor truck, and the intimate connections now existing between the railroads and industrial plants, fail utterly to neutralize the economy of the water carrier, and far more than enable the absorption of transfer charges to and from the river bank.

There is now an indisputable record of the costs of modern water transportation, although far from fully developed and equipped, in the Government service operating on the Mississippi River between St. Louis, Mo., and New Orleans, La., a distance of 1 154 miles.

Knowing that the fitful work of the Federal Government had about twelve years ago developed a permanent, navigable channel on the main stream of the Mississippi from St. Louis south, the speaker, about seven years ago, when President of the New Orleans Board of Trade, appointed a special committee to investigate the possibilities of water transportation. The old river steamers and the thousands of varied craft which, prior to the Civil War and for more than twenty years thereafter, had carried the bulk of the traffic from the interior of the valley to the seaboard, had been eliminated. A combination of uncertain channels and strenuous rail competition had prevented the construction of efficient modern carriers, so that, in 1916, except for a few barges and carriers of local traffic, the only river craft of consequence in operation were those of the Government engaged in maintaining unused navigable channels.

The investigations of the New Orleans Committee encouraged that body and the Merchants Exchange of St. Louis to approve the organization of a private corporation in which each was to invest some of its funds. Fortunately for the prospective investors, however, war prices and conditions made it commercially impossible to proceed at that time. Experience afterward demonstrated that long before proper co-ordinating relations with the railroads—an absolute essential—could be established, any possible private corporation would have been bankrupt. The entry of the United States into the World War, the operation of the railroads by the Government, and the need of additional transportation, enabled those interested to convince President Wilson of the opportunity for re-establishing effective water transportation and its co-ordination with the services of the railroads.

The service established by the Federal Government operates fleets of modern steel barges and efficient twin-screw towboats for 1 154 miles on the Lower Mississippi between St. Louis and New Orleans, and on the Warrior River and connecting waterways for 600 miles between New Orleans and the Birmingham District in Alabama. The navigable channel of the Warrior and the necessary entrance from the chain of Alabama waterways to the harbor of New Orleans have been incomplete until quite recently, and the essential joint rail and water rates with equitable divisionals, in that section, are still to be arranged. By these a much greater territory can be served, and increased water traffic and revenue provided.

During the fiscal year ended June 30, 1922, the service on the Mississippi carried 655 000 tons of freight. The freight bill of the public would have been $850 000 greater on this traffic had not the Barge Line been available with its tender of lower rates than those published for a through all-rail service.

The financial statement for the Mississippi Section, for the fiscal year ended June 20, 1922, as reported to the Interstate Commerce Commission, is as follows:

Gross tonnage	655 789
Gross earnings	$2 785 566.00
Deductions to cover terminal absorptions	225 818.27
Gross freight revenue to cover operations	2 559 747.73
Gross operating cost inclusive of depreciation*	2 438 720.67
Remainder, applicable to return on Government investment	121 027.06

As the reproduction value of the Government fleet and facilities used by the service on the Mississippi River is from $3 500 000 to $4 000 000, the Government received a return of 3½% on its investment, after it had saved the public an average of $1.30 on every ton of freight handled.

In the first 2½ years of its service, with inefficient, makeshift equipment and terminals, as might have been expected during the formative stage, a deficit of $1 400 000 was incurred. Nevertheless, the shippers who used the service saved more than this sum by the reduced rates on more than 1 000 000 tons of merchandise. The deductions, to cover terminal absorptions, represent very liberal switching charges paid to the railroads to take the freight between the local warehouses and industries and the Barge Line terminals, for the service gives the public the same through transportation as the railroads. The accrued depreciation is not only liberal, but, by requirement of the Commission, is based on the book value of the plant—one built at the peak of war inflation. The operating costs include sums fixed by the various States and municipalities for the carrying charges on interchange terminals constructed by them for the use of this and other water lines, as well as the cost of maintenance of such terminals. The most modern and satisfactory terminals for this interchange are now constructed and in use at St. Louis, East St. Louis, Ill., Cairo, Ill., Memphis, Tenn., and New Orleans. A new terminal is being constructed at Vicksburg, Miss., and the municipal terminal at Memphis is being greatly enlarged. More than $6 000 000 has been invested in these terminal plants, largely by the cities included in the service, in the years since the Federal Barge Line was started. In its total operating cost for the last fiscal year, the Mississippi Section paid out $118 889 for carrying charges and the maintenance of these terminals.

It should be noted that, whereas the reports of the Interstate Commerce Commission show that, on Class I railroads (about 90% of the mileage), the ton-mile earnings in 1921 average 1.275 cents, that of the Barge Service on the Mississippi for the twelve months ending June 30, 1922, was 0.388 cent.

* Depreciation included $265 159.89.

The Mississippi-Warrior Service has joint traffic relations with all the railroads in the Mississippi Valley and the Southeast. On the Mississippi Section more than 60% of its traffic (consisting of general merchandise and grain) is joint traffic, in which the railroads participate, that is, the bulk of the freight carried by the Barge Line has either its origin or its destination at railroad stations in the interior, away from the river bank. Thus, the co-ordination of this Federal Barge Line with the railroad system of the country is already in part an accomplished fact.

One change brought about by the Federal Barge Line is that the farmers have a cheaper outlet to the sea for their grain. The line is frequently taxed to its capacity, carrying from 2 000 000 to 3 000 000 bushels of wheat and corn each month to the seaboard; and the saving of about 3 cents a bushel in its rates is being reflected in the price paid for this produce in the interior.

On every class of merchandise, the shippers of Ohio, Indiana, Michigan, Illinois, Wisconsin, Minnesota, Iowa, Missouri, and other Mississippi Valley States have a cheaper route to the sea than the all-rail route, and every kind of package freight produced in this great territory is seeking an outlet through the Barge Line. One manufacturing company, not near the Mississippi River, has saved more than $60 000 in 10 months on commodities brought from the Orient and distributed throughout this central territory. The river rates, based on the tariff rail rates between the points of river operation, are made 20% less than the rail rate. It is important to state that, although this service does not extend north of St. Louis, shippers at many interior points, remote from navigation—Fargo, Minn., Warren, Ohio, Fort Wayne, Ind., Lincoln, Nebr.—and residents of the great territory named have offered to them, by the use of rail and water co-ordinated service, the same saving in their freights as if they lived at St. Louis or other stopping points of the service on the Mississippi River. For the first time in the history of the United States, these great manufacturing and agricultural districts, which hitherto have had no direct interest in the economy of water transportation, are placed on a parity with those on the river banks.

It is the desire of the management of this Barge Service to obtain, from the Interstate Commerce Commission and the railroads, authority to enlarge still more the joint rail-and-water territory, so as to include every city and every shipper who can use it advantageously. The revival of commercial transportation on the Mississippi and Warrior Rivers is now established beyond question. Public welfare demands a vast increase in this great facility, and it should come by the desirable and natural process of ordinary commercial enterprise. The rivers belong to the Nation, are controlled and improved by its Government, and all citizens, equally, are entitled to service. Only by agreement with the railroads can river traffic reach beyond the banks. To confine the economies of the river in this way not only denies them to the infinitely greater citizenship remote from the banks of the river, but also condemns to failure the effective service of water transportation. As long as the roads fail to agree voluntarily with the water lines, and impose objections which must be considered and decided by the Interstate Commerce Commission, it is evident that the delay

in adjudicating joint-traffic arrangements is beyond the economic possibility of commercial enterprise. No private concern can afford to wait twelve months or two years for decisions as to rate territory and divisions, with its floating equipment even partly idle and overhead expenses accumulating daily.

Much, therefore, as Government operations in transportation or any other genuine business activity may be disliked, two things are quite evident: First, that no possible commercial enterprise can meet successfully the present difficulties in joint rate arrangements; and, second, that until the railroads show a definite change of policy, and a real willingness to assist in furnishing the public with the comparative economy of joint rail and water service, the Government must remain in this transportation business, or the country must be denied one of its most urgent necessities and one of its most valuable transportation auxiliaries.

THE ECONOMICS OF TRANSPORTATION ON THE MISSISSIPPI RIVER

By Joseph E. Ransdell,* Esq.

The speaker is pleased to appear before the American Society of Civil Engineers to discuss this interesting subject. Since entering Congress, 24 years ago, he has specialized on legislation relating to rivers and harbors, and has had closer connection with Army engineers than with officials of any other class. Moreover, the Mississippi and the mighty problems of the control of its floods, from which the speaker's home section frequently suffers, have kept him in intimate touch from his youth with levee engineers of State and Nation, and with the Mississippi River Commission.

Great Importance of Transportation.—There is no more important or difficult problem in America than transportation. It affects, in a personal way, every citizen of the land. It affects materially the food we eat, the clothes we wear, the houses that shelter us, and the carriers that convey us from place to place. It is such a common, everyday thing, that most of us do not realize its meaning. When transportation is plentiful and rates are reasonable, all is well and the people are happy, but there have been times when the reverse was true and the resultant suffering very severe.

The speaker is an optimist by nature and training, but is bound to admit the evidence of plain facts; and no one who examines the subject carefully can deny that transportation is a most serious problem. The following is quoted from three able authorities.

Mr. Alfred P. Thom, General Counsel of the Association of American Railway Executives, stated to the Senate Committee on Interstate Commerce in 1916:

"Have we learned no lessons from what happened in 1907, when there was a substantial increase in the business offered to the railroads, and lack of yards, lack of tracks, and lack of cars brought on the 'panic of plenty' in that year? Have we forgotten that the panic of 1907 was not a panic of scarcity, not a panic of failures in business, but was a panic brought on by the inability of communities to deal with one another because the railroad facilities were inadequate? Congestion everywhere; not yards of sufficient capacity for trains; not tracks sufficient to carry them; not cars sufficient to transport the business of the people. There, in that year, in the midst of that plenty, came panic, due to those factors."

Mr. Elisha Lee, Vice-President of the Pennsylvania Railroad, before the Manufacturers' Association in Philadelphia, Pa., in 1922, made the following strong statement which stands unrefuted:

"Traffic on our American railroads, measured in ton-miles, doubles about once in a decade, or possibly a little longer. This rate of increase has been maintained for at least two generations with surprising regularity, despite the various cycles of booms, panics, and depressions through which the country

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has passed meanwhile. The events of the war, of course, pushed our traffic figures up to levels some years ahead of the normal growth, and we are now having a corresponding reaction, but, get this fact firmly in your minds, because it is most important.

"The next time our country has a real revival in business we shall, in all probability, be confronted with the most severe congestion of railroad traffic, and the greatest inadequacy of railroad facilities, ever experienced in our history. When that happens, rates will be lost sight of. Every one will be clamoring for service, and our public highways will again be torn to pieces by huge truck loads of freight, carried over roadways never designed for such purposes, and at rates and costs of operation so high as to constitute gross economic waste. Nothing would more quickly check a wave of prosperity than the inability of our railroad facilities to handle the traffic which good times would create.

"I am firmly convinced that we face such a condition with almost absolute certainty, in the not remote future. Whether it will come within a year or two, or be deferred for several years, I do not for a moment assume to predict, but, unless the world plunges into hopeless economic chaos, in which case all reckoning will be wrong, we are going to push up our national figures for production, and consequently of potential railroad traffic, beyond anything ever known before in peace times, and in all likelihood even beyond the levels reached during the war. Then business men will not be bothering themselves very much about rates. All they will be thinking about will once more be how to get transportation at any price.

"For this situation errors of the past are to blame. We cannot undo the past, but we can learn from it, and while probably nothing can be done now to obviate altogether the coming period of railroad congestion, which all railroad officers foresee, we can at least mitigate its severity and address ourselves to the task of assuring adequate transportation facilities for the more distant future."

The most impressive quotation is the solemn note of warning from one of the nation's ablest men, Herbert Hoover, M. Am. Soc. C. E., Secretary of Commerce, who stated in his latest Annual Report:

"Our transportation facilities have lagged far behind the necessities of the country. * * * Except during periods of business depression or strikes there has to some degree been continuous car shortage for the last six years. Furthermore, car shortage reaches its most acute state during the four or five months of peak load in the fall and early winter. The losses through shortage of transportation are a tax upon the community greater than the cost of our Government. * * * There can be no question that action in some direction is imperative, if industry and commerce are not to be further strangled by a shortage in transportation. * * * We must have increased transportation, if we are to maintain our growing productivity."

The speaker agrees absolutely with these gentlemen. "Railroad facilities were inadequate" in 1907, as stated by Mr. Thom, and they are far more inadequate to-day. Samuel Rea, Hon. M. Am. Soc. C. E., President of the Pennsylvania Road, said recently that the total length of the railroads in actual operation in the United States had decreased 300 miles during the last 10 years. President Markham, of the Illinois Central Railroad, stated some months ago that, during the past 7 years, the annual increase in tractive power has been less than one-half, and in freight car capacity only about one-fifth of what it was 14 years earlier.

All economists seem to agree that the demands of transportation increase practically 100% every 10 years. Mr. Lee is correct in predicting that in the near future "we are going to push up our national figures for production, and consequently of potential railroad traffic, beyond anything ever known before in peace times", and when that day comes, "business men will not be bothering * * * about rates", but, "how to get transportation at any price."

Secretary Hoover never uttered wiser words than when he said, "The losses through shortage of transportation are a tax upon the community greater than the cost of our Government", and, "We must have increased transportation, if we are to maintain our growing productivity."

Annual Loss from Shortage in Transportation.—The cost of the Government for the fiscal year 1922 was $4 000 000 000, in round numbers; therefore, if Secretary Hoover's estimate is right, the losses inflicted on the agriculture, the commerce, and the industry of the United States by shortage of transportation have amounted to more than $24 000 000 000 in the past 6 years. This huge sum equals the total cost, to the United States, of the World War and its subsequent expenses to date, and exceeds by some $4 000 000 000 the entire capitalization of the railways of the United States. It is unthinkable that such staggering losses should be allowed to continue if any possible way can be found to prevent them.

Relief from Waterways.—It is the speaker's firm conviction that a large part of these losses, depicted so vividly by Secretary Hoover, and presaged so eloquently by Mr. Thom and Mr. Lee, can be prevented by the wise use of the Nation's waterways in friendly co-operation with highways and railways. Although it is true that boats, to a great extent, have disappeared from many of our rivers, there is still a colossal water-borne commerce which can be increased materially and rapidly if the rivers and railroads work together as auxiliaries and friends, instead of fighting.

The water-borne commerce, both foreign and domestic, of all the rivers and harbors of the United States in 1920 amounted to 369 269 026 tons, valued at $19 510 496 495. The domestic freight handled on the waterways that year—not including ferry traffic—amounted to no less than 256 496 113 tons. Counting 30 tons to a car, this is equal to 8 550 000 carloads, an average of 712 500 per month, or 23 750 per day.

Figures for the rail tonnage for 1920 are not at hand, but, according to the Interstate Commerce Commission, the total reported as carried in 1921 by the Class I roads, excluding the duplications resulting from interchange between carriers, was 940 182 560 tons. It appears, therefore, that the waterways are carrying almost one-third as much of the domestic commerce as the railways—surely a very respectable percentage.

Freight Possibilities from Fully Improved Rivers.—If the entire 38 000 miles of nominally navigable streams in the Union were properly improved and made available for boats of suitable draft, it would be difficult to estimate the volume of freight that could move thereon. It would certainly exceed by 200 to 300% the volume now carried by water. Many of the waterways are in process of improvement, and public sentiment seems to favor a liberal policy toward them. Such a course would be extremely wise. The sums neces-

sary to finish most of the important projects on rivers and coastal canals would be small in comparison with the several billions needed for betterments and new mileage of the railroads, and would give very large returns in much needed transportation facilities. The speaker hopes for the speedy improvement of these watercourses, and their beneficial use in helping to solve big freight-moving problems.

Rivalry Between Rail and River.—During many years, boat and rail transportation fought in bitter cut-throat competition, with the result that the immense water-borne commerce of the period prior to the Civil War diminished rapidly in the 40 years following, and had almost ceased to exist on some streams at the end of the Nineteenth Century.

The railways were able to serve all sections, but boats could reach only a limited area near the water. This gave the railways great advantage. They could make extremely low rates at river points and recoup losses thereon by highly profitable charges at interior places. The boats were unable to compete, and many of them ceased to operate. Sometimes a boat would start on a route not served directly by rail, and the nearest railroad would put in a competing, or pirate, boat to drive out its rival by unprofitable rates, after which this dishonest emissary would tie up indefinitely. Those were palmy days for railroads, and sad ones for boats. Now, the railroads are in trouble also, and the boats are beginning to come back slowly.

During the World War the railroads were taken over, and operated, by the Government. Railroad men claim that they have not yet returned to a paying basis, on account of the many restrictive laws and regulations to which they have been subjected. Moreover, there is much dissatisfaction with high freight rates and inefficient service. Many are looking to Government ownership and operation of railways as the panacea of these ills. This is true, not only with a large class of citizens who have suffered, or imagine they have, from railroad shortcomings, but also with holders of rail securities, whose investments are so unsatisfactory that they would gladly have the Government purchase the roads. It is said that, if the Government would take the roads and pay for them the value assessed by its experts, the owners of the bonds and stocks would fare much better than is indicated by present returns on these investments. This would be most unfortunate, for many reasons.

Government Ownership and Operation of Public Utilities.—The Government should not do anything which can be done as well by its individual citizens. Free republics should refrain from interference with all private business which is conducted with due regard to the rights of others.

It may be said that the public owns the waterways and highways, why not the railways? A fair reply is that there is no monopoly on either highway or watercourse. Both are at the service of every citizen on equal terms, and are used by all who so desire. It is not practical to operate a railroad, save as a monopoly, either by a citizen or the Government itself. No workable plan has yet been devised for the public ownership of railroads coupled with their general private use, hence there is no comparison between the ownership, by the Nation, of the highways and waterways on the one hand, and railways on the other. It would be thought an unbearable hardship if only Government

vehicles could use the highways, and Government boats the waterways. It is necessary either to continue the present system of private ownership and operation of railways, which on the whole has been satisfactory, or make a complete change and have them both owned and operated by the Government. There is no middle ground.

Friendship Between Rail and River.—The majority of the people are opposed to Government ownership, and most of the big railroad leaders would fight it vigorously. There is no more effective weapon for this purpose than to increase facilities and cheapen transportation of all kinds by genuine friendship between road, rail, and river. If the old-time bitter competition is to continue, and facilities are to diminish and become higher priced, it does not require a prophet to predict Government ownership and operation of railroads in the near future. Such a calamity can and must be prevented.

The transportation watchword must be cordial co-operation between highways, railways, and waterways, in order that the public may receive the best service at the least charge that gives fair returns to carriers. Unfair competition must cease. It is fatal to all concerned—the carriers and the people. Let there be no more of it!

Railways should treat rivers and canals as brethren in the Transportation Trinity, and *vice versa,* for the cut-throat business was not all on one side. There is no necessity to show favors to boats. Merely give them the same treatment, and the same equitable division of freights to which a railroad performing the same service would be entitled; establish joint rail and water rates with through bills of lading, assigning to each carrier its proportionate share of the freight money based on service rendered; and allow each carrier to have a fair share of the business offered.

In other words, let the railways recognize the rivers as a real part of the transportation system, and apply to all their dealings with them the principles of ordinary honesty and justice. If service can be rendered satisfactorily by boats, do not drive them out of business by unprofitable rail rates at river points, offset by much higher rates at interior places, as was done in the past. The shipper whose home is a hundred miles from the Mississippi is entitled to the same rail rate for equal mileage as one who lives on its banks. If railroads would recognize this fact and apply it fairly to the interior rivers, just as they do to the Great Lakes and the oceans, the long trouble between river and rail would speedily adjust itself.

The speaker has never heard of any serious rate competition between carriers on the Lakes or seas and the railroads. There is a colossal Lake commerce which is of great importance to the nation. It was carried last year at an average cost of 1.2 mills per ton-mile, which was about one-tenth of the average cost of rail transportation for the country as a whole. There was co-operation, not competition, between the Lake boats and the railroads. The same situation must be brought about between the rivers and the railroads.

There should be a new understanding between railways and waterways. Both should forget the errors of the past; profit by their hard experience; work together in the future, each in its own field as far as may be; carry at a profit the freight that comes to it, and only such as pays a profit directly or indi-

rectly; eschew everything that borders on unfair competition; and do whatever is possible to furnish efficient transportation to every citizen on equal terms at low rates. If that policy is adopted, not only by water and rail carriers between themselves, but also toward motor trucks, there will be a wonderful improvement in transportation facilities.

The Mississippi River System.—The Mississippi System of rivers affords incomparably the most extensive inland waterways in America. The navigable parts of the system reach twenty-one States of the Union. The area of the basin is about 1 240 000 sq. miles.

Above the Passes at its mouth, the Mississippi has a natural depth sufficient for deep-draft ocean vessels to above Baton Rouge, La., 250 miles from the Gulf. New Orleans, more than 100 miles from the outlets of the river, is the principal transhipping point between river and ocean transportation.

From the Head of the Passes to Cairo, Ill., a little more than 1 000 miles by river, the Mississippi River Commission maintains a channel 9 ft. deep at all times, except when, at comparatively rare intervals, under particularly unfavorable conditions, the dredging fleet is unable temporarily to dredge fast enough to keep channels of full dimensions open through all the bars that form. This section of the river is navigable throughout the year.

From the mouth of the Ohio to St. Louis, Mo., 183 miles, a depth of 8 ft. is maintained. From St. Louis to St. Paul and Minneapolis, Minn., 686 miles, there is a project to secure a 6-ft. channel. The present limiting depth at low water in the shoalest section is 4 ft. at the Rock Island Rapids.

The most important tributary of the Mississippi, with regard to navigation, is the Ohio, which is under improvement from Pittsburgh, Pa., to Cairo, 968 miles, under a project providing for 9-ft. slack-water navigation by the construction of locks and dams. The project is well advanced, and is being prosecuted actively, with good prospects of being finished in 5 or 6 years. When completed, a navigable waterway, nearly 2 000 miles in length, with a dependable 9-ft. depth, will be available from the great industrial region about Pittsburgh—the largest freight-producing center on earth—to the seaport of New Orleans. Practically all the principal tributaries of the Ohio have been, or are being, improved by canalization and open-channel work, as feeders of commerce to the main stream.

The State of Illinois has embarked on a project for a waterway with 9-ft. draft, to connect Chicago, Ill., with the head of navigation on the Illinois River. The present depths in the Illinois River, and in the Mississippi between the mouths of the Illinois and the Ohio, are less than 9 ft. It is to be presumed, however, that an adequate connection between the "Illinois Waterway", as it is called, and the Mississippi will be authorized in due time, and the 9-ft. channel extended to Cairo. This will afford facilities for a colossal barge traffic between the Great Lakes and the Mississippi River System, with a dependable depth of 9 ft.

The longest navigable tributary of the Mississippi, but the one on which it is most difficult to provide adequate navigation, is the Missouri. A project has been authorized, however, for a 6-ft. channel from the mouth to Kansas City,

and, when it is completed, that important distributing point will have access to the Mississippi.

With these and other tributaries and their branches, the Mississippi System will embrace about 14 000 miles of navigable waterways. Channels aggregating 1 580 miles now have a dependable depth of 9 ft., and nearly 2 000 miles will have this depth when existing projects are completed; 3 500 miles of channel now have a depth of 6 ft., or more at low water, and 4 000 miles are to have at least that depth. The remaining parts have depths of less than 6 ft. at low water. Many of the latter channels, however, have ample depths for navigation for a considerable part of the year.

Development of Commerce on the Mississippi.—Professor Dixon, of Dartmouth College, has written a "Traffic History of the Mississippi River System",* and the Annual Report of the Mississippi River Commission for 1901 contains, in Appendix A, a paper by John A. Ockerson, Past-President, Am. Soc. C. E., giving a study of the traffic from the end of the Civil War to 1900. The report of the Chief of the Inland and Coastwise Waterways Service, for the fiscal year 1920, gives the history of the revival of water transportation. A brief summary of the data in these papers, in the reports of the Chief of Engineers, and from other sources, follows:

The navigable rivers of the great interior valley afforded the routes by which this extensive territory was settled and developed. In early days the only other means of transportation were unimproved earth roads. The products of the country necessarily moved on the rivers to an outlet to the sea at New Orleans.

The earliest navigation was by relatively small craft maneuvered by hand, many of which were roughly constructed for use on a single down-stream trip, and intended to be broken up on arrival at their destination. Steamboats were introduced in 1811, and, after a period of development, increased rapidly in number. The annual arrivals of steamboats at New Orleans increased steadily from 21 in 1814 to 3 566 in 1860.

Up to a decade before the Civil War, the construction of railroads had not developed to such an extent as to draw any traffic from the river. On the contrary, the early railroads were constructed as feeders to the river traffic system. In the last decade before the Civil War, the river traffic, although still increasing, showed some slackening in its rate, presumably on account of diversion of the traffic by the railroads. By 1860, the value of the annual receipts at New Orleans had reached $185 211 254. The tonnage at New Orleans in 1840, the last year prior to the Civil War for which figures are given, was 542 500, with a value of $49 822 115. The tonnage in 1860, if it increased in the same proportion as the value, must have been about 2 000 000.

New Orleans Fourth Port in World in 1840.—Professor Dixon has stated.†

"New Orleans, at the terminus of river transportation, grew with rapidity, and was rated in 1840 as the fourth port in point of commerce in the world, exceeded only by Liverpool, London, and New York."

* Document No. 11 of the National Waterways Commission, December, 1909.
† "Traffic History of the Mississippi River System", p. 15.

The Civil War brought the river commerce on the Lower Mississippi to an abrupt termination, and, by closing the natural outlet of trade, crippled the entire system. When the river was re-opened, after the war, river transportation had to face the period of rapid railroad expansion, and unregulated competition. It was hampered in the struggle by the fact that the channel for ocean vessels to the Port of New Orleans had not been improved to meet their increasing draft. The Passes of the Mississippi had a natural depth sufficient for the passage—often with some difficulty—of the sailing vessels of pre-war times, but, with the change in size and build of vessels that came with the transition from sail to steam, the shallow entrances became impracticable. It was not until the completion of the improvement of the South Pass, in 1879, that New Orleans regained its position as an ocean port.

Notwithstanding these difficulties, river traffic was resumed. A marked change, however, took place in its character. Before the Civil War the dominant feature was the packet steamboat. After the war, although packets remained, especially in the local trade, the dominant characteristic was the fleet of barges handled by a powerful towboat. A barge line was organized as early as 1867 to carry grain from St. Louis and other upper river points to New Orleans. Despite the handicaps noted, the line could offer rates which enabled shippers to export to New York City at lower prices than by rail. Although the railroads countered with decreasing rates, the barge line was able for many years to meet them and maintain the business. This traffic reached its zenith in the Eighties. As many as 15 000 000 bushels per annum, or about 30% of the grain shipments from St. Louis, were moved by river.

Another and much larger traffic element was developed in the transportation, on high-water stages of the Ohio, of coal from the Pennsylvania and West Virginia fields to New Orleans and other lower river points. Even prior to the Civil War, large shipments were sent down river from Pittsburgh, the quantity in 1860 being a little more than 1 000 000 tons. The traffic increased rapidly after the war, and by 1888 somewhat more than 4 000 000 tons passed down river from Pittsburgh. From 1 000 000 to 2 000 000 tons were carried in tows of great size, handled by especially powerful towboats. The towboat *Sprague* once took to New Orleans 56 000 tons of coal, the greatest volume in a single cargo ever moved on any waters.

As long as the forests in the States bordering the Upper Mississippi and its tributaries continued to yield lumber, rafts formed another large element in the commerce of the river.

Steady Decline in River Traffic.—A decline in river traffic, beginning in general some time prior to 1900, progressed steadily until 1917, the year the United States entered the World War. The grain shipments from the upper river dropped off sharply in 1903, and had reached an insignificant figure by 1917. Coal shipments from the Ohio River remained at 1 000 000 tons or more annually until 1915, after which they declined rapidly to about a quarter of that figure. This decrease in coal was due to its absorption in the producing territory, and the development of shipments to New Orleans from the Southern coalfields. Omitting the local traffic in sand and gravel dredged from the river

for building purposes, an industry that reached large proportions in some localities, traffic on all parts of the Mississippi, except between New Orleans and Vicksburg, Miss., declined to a low ebb. Even in this section, however, the relative proportion of the traffic handled by water steadily decreased.

As a result of this decline, the steamboats and barges gradually disappeared. The traffic policy, under which rail carriers had made undue reductions in freight rates to meet potential as well as actual water competition, had made the investment of capital in river craft unduly hazardous. For years no new boats were built to replace those worn out or destroyed; no advances were made in the type of craft or in the labor-saving devices for loading and unloading freight; no efficient water terminals were constructed; and commercial steamboating on many of the rivers was practically dead until revived by the World War.

In the meantime, rail carriage had progressed rapidly, with the aid of more perfect roadbeds, heavier rails, larger cars and locomotives, and modern terminals. It is proper to add that commerce on the Great Lakes had kept full pace with the railroads, and perhaps surpassed them in its wonderful devices for handling freight to and from vessels and cars with great celerity and very small cost.

Revival of River Traffic by the World War.—The spurt of war preparation in 1917 strained rail transportation to the breaking point. In the autumn of that year traffic conditions became so acute that the Board of Trade of New Orleans, in conjunction with the Chamber of Commerce of St. Louis, undertook the creation of a corporation to operate a barge line between these cities. It was necessary to build a fleet for that purpose, but the necessary steel could not be obtained except for war purposes. Finally, the efforts of these commercial representatives resulted in the belated construction of a fleet by the United States Railroad Administration, which was subsequently turned over to the Inland and Coastwise Waterways Service, under the provisions of the Transportation Act of 1920. The operations of this barge line have restored the Mississippi to its former position as a commerce carrier, and the outlook for the line is promising.

The revival of water transportation has not been limited to these Federal enterprises. The construction of modern towboats and barges to meet the shortage of available means for water transportation, particularly on the Ohio and its tributaries, has been actively undertaken. The large steel producers in the Pittsburgh District are manifesting great interest in the transportation of their products by water down the Ohio and Mississippi, and have already started such shipments during high-river stages.

Lack of River Terminals.—The lack of suitable terminals for handling freight by river and effecting its interchange with trucks and cars has been one of the causes, assigned by students of the economics of water transportation, for its decline. It is apparent, however, that as long as water transportation was steadily declining, as the result of low rail rates to river points offset by high rates to places away from the river, and because the railroads placed every obstacle in the way of interchange of rail and water transportation, capital could not be attracted to the construction of the expensive terminals

necessary to meet the inherent difficulties caused by such large fluctuations in water level as are common on the rivers of the Mississippi system. With the establishment of the Federal barge line on the Mississippi, and the revival of the interest in water transportation, terminal construction has become active. Several river cities have constructed, or are now constructing, satisfactory terminals. This movement for effective terminals will spread rapidly if the plan of cordial co-operation between highways, railways, and waterways is adopted and carried out in good faith.

Costs of Transportation on Rivers and Lakes.—The cost of river transportation per ton-mile is not a fixed quantity. It varies within quite wide limits, depending on the depth of the available channel and its width, and according to whether the traffic is up or down stream. It is needless to say, also, that to the operating cost per ton-mile must be added the cost of delays at terminals, and the general overhead of conducting the business.

River transportation in the United States has reached its highest development on the Monongahela River. This river has been improved by the construction of locks and dams to afford an 8-ft. slack-water channel through the section carrying heavy commerce. Figures compiled recently by the District Engineer at Pittsburgh show that the cost of moving this commerce ranges from a minimum of 1.72 and 1.98 mills per ton-mile through an average of 2.58 to 3.26 mills, for hauls of 56 and 65 miles, respectively. These costs include interest and depreciation on the equipment at 6% and 10%, respectively, as well as operating costs, but do not include the terminal charges at the point of receipt or delivery. The unloading facilities are developed so that delays at the terminals are brief. Present rail rates between the respective terminals are 18 and 17.3 mills, respectively, per ton-mile of haul, based on river distance. In other words, average transportation costs on this river are about one-fifth of the cost of rail transportation.

A series of tests by the Experimental Towboat Board, from 1916 to 1920, under an investigation ordered by Congress, gave operating costs of 0.8 mill to 4.1 mills per ton-mile, exclusive of terminal and fixed charges for traffic on the Mississippi above Cairo (8 and 6-ft. open-river channels). The tests were conducted under commercial conditions, but the towboats used cannot be regarded as being as efficient as those which would be provided at present for such service. The average cost was approximately 2 mills per ton-mile. Data received by the Board from five commercial transportation companies indicated that the cost varied from 4.3 mills in 1912 to 6.39 mills in 1919, these figures evidently including fixed charges and profit.

An elaborate study of and report on the Ohio River was made in 1906 by a special Board of Army Engineers, which strongly urged the 9-ft. project on the river. This project was adopted by Congress in 1910, and is now approaching completion. The Board gave, as an estimate of the cost of conveying coal in large quantities from Pittsburgh to New Orleans on the 9-ft. channel, 0.5 mill per ton-mile, stating: "This price includes maintenance and 5% interest on investment, and is about one-eighth the average rail rate on similar commodities."*

* House Document No. 17, 60th Cong., 1st Sess., p. 18.

The statistical report of Lake commerce passing the canals at Sault Ste. Marie during the season of 1922 shows that more than 66 000 000 tons were carried an average of 810 miles, at a cost of 1.2 mills per ton-mile. In 1913 this commerce was more than 85 000 000 tons, and the rate was much less than 1 mill per ton-mile. The average cost of this Lake commerce for 20 years, including costs of loading and unloading, was 0.82 mill per ton-mile.

The Mississippi Barge Line, operating with efficient equipment, carries freight profitably, on joint through bills of lading with railroads, to large sections of the valley, many miles away from the river, at 80% of the rail charge for the same service; and the prospects of this line are very promising. Details in regard to it have been given by Mr. Sanders.*

From these figures it appears that the cost of transportation by water varies greatly in different localities, and it is difficult to make an exact comparison between water and rail costs. The speaker's conviction is that water is inherently much cheaper than rail, principally because the upkeep of the roadway is incomparably less and the frictional resistance of a liquid is not nearly so great as that of a steel rail. There is scarcely any limit to the potential tonnage of a stream like the Lower Mississippi, but the number of trains that can use a large railroad—the Illinois Central, for example—is decidedly limited. Moreover, the standard rolling stock of a monopolized railroad is much more expensive, in original cost and maintenance, than that of the numerous varieties of craft on a free river—a flatboat moved by the current, a skiff driven by human muscle, a gasoline launch, a fleet of barges towed by a power boat, or a palatial steamer carrying freight and passengers.

Freight Costs by Rail.—The cost of rail transportation varies with roadbed, grade, and condition of traffic, in the same manner that river transportation varies with channel facilities. The average revenue of the railroads per ton-mile, in 1922, as reported by the Interstate Commerce Commission, was 12.74 mills, varying from 10.08 mills in the Southern District to 14.22 mills in the Western District. The revenue per ton-mile of freight on the Illinois Central in 1922 was 9.66 mills for the entire road.

It is well recognized that the cost of handling cars at terminals forms a very large part of the cost of rail transportation. Cars must often be hauled many miles to and fro over terminal tracks to be segregated and delivered at their unloading destination. River barges are not bound down to rails and switches, but may be moved directly to their unloading berths. The terminal costs of water transportation, therefore, are usually far less than those of rail transportation.

Ton-mile costs, admittedly, do not provide a correct comparison between rail and water transportation, because the water routes are generally much the longer. Thus, the distance from St. Louis to New Orleans by the winding course of the river is 1 151 miles, but, by the most direct rail route, it is only 718 miles. From Pittsburgh to New Orleans by river is 1 936 miles, but the most direct rail route is 1 147 miles.

On the other hand, the rail rates between two river points do not indicate the cost of the rail service, meaning by cost the average sum that the railroads

* See pp. 2024–2026.

must receive, in order to secure a fair return on their investment. It is a matter of common knowledge that, to meet river competition, rail rates between points having water competition have been reduced below the rates in force for non-competitive routes. Interior points, therefore, pay an undue proportion of the total cost of the transportation service.

A study, published in the report of the Chief of Engineers,* shows that the rail rates on bituminous coal are generally from 10 to 20% lower where there is water competition than for similar hauls without such competition. The difference between rail rates to points where there is water competition and the rates for an equivalent haul where there is no water competition, is often much more than 20 per cent. Thus, the rates recently established on first-class freight between Minneapolis and St. Louis is \$1.06½ per 100 lb. for a 586-mile haul, or 36.2 mills per ton-mile. The rate from Minneapolis to Kansas City, a distance of 500 miles, is \$1.44 per 100 lb., or 57.6 mills per ton-mile. The non-competitive rate in this case, therefore, is nearly 60% in excess of the rate made between points subject to water competition.

Big Shippers Use Waterways.—The final determination as to whether rail or river is the cheaper method of transportation must rest with actual commercial experience. It is enough to state that, where freight is offered in sufficiently large quantities, and the river channels are adequate, it actually moves by water. The great steel manufacturers of Pittsburgh receive their coal by water from the mines on the Monongahela, their ore by water and rail in friendly competition from Minnesota, and find it profitable, even in the incompleted state of the improvement of the Ohio, to send, when they can, their manufactured product down the river in barges.

The *Times-Picayune* of March 26, 1923, recorded the arrival in New Orleans of a towboat of the Carnegie Steel Company, from Pittsburgh, with six barges carrying 5 000 tons of iron and steel products, half of which was destined for shipment by rail to Texas. At about the same date, the Jones and Laughlin Steel Corporation, of Pittsburgh, shipped from there a tow of eight steel barges with 7 000 tons of steel and iron products. Three barges were dropped at Louisville, Ky., one at Evansville, Ill., one at Cairo, and three went to Memphis, Tenn. That freight was all transhipped by rail to points in eleven States.

There is no doubt that the commercial interests of the river cities along the Mississippi regard the barge line established by the Federal Government as a commercial necessity. This line has demonstrated beyond a doubt that, when properly handled, it is a money maker for the Government and a money saver for the public. By it, the shipper, on the average of the long and short hauls, saves \$1.35 a ton. The line has come to stay. There is an increasing demand for the facilities offered by it. The public needs these facilities and will insist on having them.

Co-operation between the barge line and the railroads is growing. The fundamental difference between the line and the railroads can be adjusted. It is merely a question of division of revenue for service performed. It is not whether the river can perform the service more economically, but whether the

* Annual Report for 1921, Part II, p. 22.

people who have been taxed for its development should have the benefit of its transportation facilities. If the officials of the railroads and the barge line would compute a fair basis on which they could divide the joint revenue earned by them, there should be no dispute between them.

Even if waterways in general are not able to function at less than all-rail rates—which no waterways man would ever admit—they should operate and furnish such facilities as they can. It is not only cheaper transportation that is desired, but more and yet more transportation.

Conditions Essential to Success of Water Transportation.—In order that waterway transportation may be a success, there are certain conditions which must be fulfilled:

a.—There must be a suitable waterway; that is, one with a channel of sufficient depth and width at all times.

b.—There must be suitable terminals. To a large extent, these will depend on the classes of commodities handled, and the conditions under which the waterway is operating.

c.—There must be suitable and sufficient floating equipment.

d.—There must be sufficient balanced freight both ways; that is, freight on which the carriage rate runs from the highest to the lowest, giving an average revenue per ton-mile sufficient to offer a fair profit over the gross cost of operation.

e.—There must be an equitable adjustment of rates between highway, railroad, and water carriers, when freight is interchanged; and there must be such interchange of freight.

Most of these essentials already exist on the Mississippi below Cairo, and the others are being reached so rapidly that fulfilment in the near future is assured.

Statement of Waterway Expenditures.—It has often been asserted that comparison between rail and water transportation is unfair, inasmuch as the railroads must meet the cost of construction and upkeep of their lines, while the Federal Government constructs and maintains the waterways without charge to the boat lines. The cost of improving and maintaining channels in navigable rivers is greatly exaggerated. In some cases the total expenditures on the waterway, from the first appropriation made for it, have been presented as the capital cost of the improvement, without giving any credit for the annual movements of freight on the stream. It is all debit and no credit. Any system of transportation could be shown to be inordinately expensive if this method of accounting were followed. It is needless to state that no regulating body would countenance the establishment of rates on such a basis.

The total expenditures cover the original cost of the project, and the expense of maintaining the channel for all the years in which it has been maintained; and, if all these items are to be included in making up the cost, it is only fair to credit the project with a reasonable amount of earnings for the commerce carried on it during the entire period. What would be thought of a statistician who computed the capital cost of a railroad by adding all the items of its original construction, its replacement when worn out, betterments

of every kind, maintenance of way, etc., during the existence of the line, and who did not give any credit for the road's net earnings as an offset, at least in part, to these capital costs? Such a method of computation would be entirely fallacious, but no more so than that followed by those who adopt exactly that system in calculating the cost of river and harbor improvements.

These persons state that the Government has spent on its waterways—all of them, lakes, rivers, canals, and ocean harbors and channels—since the birth of the Republic, $1 062 000 000. They make no mention of the large business carried by the rivers in the early days, and their present considerable tonnage; nothing is told about the marvelous commerce on the Great Lakes of more than 100 000 000 tons every year; nothing is said of the colossal volumes of freight handled annually by the sea-coast harbors in domestic and foreign trade. To be fair, they should add that, for the improvement, maintenance, and operation of harbors on ocean and Gulf coasts, including Alaska and the island possessions, the Government has spent $368 000 000; on coastwise channels and intracoastal canals $14 000 000; on the Great Lakes $141 000 000; on rivers and lakes, other than the Great Lakes, exclusive of flood control, $443 000 000; on levee construction and flood control on the Mississippi and Sacramento Rivers $55 000 000; on investigations, surveys, etc., $21 000 000; and on projects that have been abandoned or become obsolete $20 000 000; a total of $1 062 000 000; of which sum $320 000 000 was spent for maintenance—considerably more than one-third for annual maintenance and flood control.

Careful scrutiny of these figures is invited. They were furnished by the Chief of Engineers, and the speaker suggests that hereafter an entirely different method be adopted in computing National expenditures for waterways. As one striking illustration, it may be said that, although the total expenditures on the Great Lakes in improving connecting channels, enlarging harbors, and maintenance, have been $141 000 000, the money saved to the people of the Nation in reduced freights, because of these expenditures, is more than double that sum every year.

Cost of Maintaining Channel of Mississippi and Other Rivers.—The cost of maintaining the channel in the Mississippi from Cairo to the Head of the Passes for the year 1921 was:

Expenditures for operation of dredging fleet, and for its repair, maintenance, and upkeep, including all overhead and stand-by costs, but not including charges assignable to capital account		$461 130.92
FIXED CHARGES:		
Present value of fleet	$2 250 000	
Present value of shore plant	510 000	
Total capital value	$2 760 000	
Interest at 4%		117 300.00
Depreciation at 5%		138 000.00
Total annual cost		$716 430.92

The ton-mileage by sections of the river was:

Cairo to Memphis.....	134 618 000
Memphis to Vicksburg.....	198 098 000
Vicksburg to New Orleans.....	1 031 904 000

The ton-mileage between Vicksburg and New Orleans includes the deep-draft traffic between New Orleans and Baton Rouge. Taking the ton-mileage from Vicksburg to New Orleans at only 250 000 000, the total ton-mileage for which the river channel work is properly assignable becomes 582 716 000. The total cost per ton-mile for the upkeep of the channel, therefore, is 1.2 mills per ton-mile. On the Monongahela, a river in which the traffic is well developed, the cost of providing the facilities for navigation, including interest at current rates, and a fair allowance for depreciation, amounts to approximately 1.4 mills per ton-mile. On the Ohio, and on the Mississippi between St. Louis and Cairo, the annual cost runs from 4 to 5 mills per ton-mile, but on neither of these rivers is the volume of traffic yet developed to the extent that the scale of improvement now justifies, or will justify when the improvement is completed. It is very clear that, from the economic viewpoint of the country as a whole, the cost of providing channels will not be excessive, provided these channels develop an adequate commerce.

Assessment of Maintenance Cost on Rivers and Highways.—As to the assessment of these costs, it must be recognized that navigable rivers are public highways, open to every one, on which the collection of tolls directly from the users to cover the cost of upkeep offers the same economic and practical difficulties that have led to the abolition of tolls on land highways. This question is hard to solve satisfactorily. There is some ground for the complaint that rivers, which have been improved and are being maintained at heavy public expense, are used by shippers entirely free of charge.

This is not the case with highways in Louisiana and many other States. These public roads belong to the States and their political subdivisions. They are built and maintained by State and local funds, aided by large contributions from the Federal Government for roads of certain classes. Every one is permitted to use them on equal terms. In Louisiana, and in several other States, automobiles and trucks are required to pay annual license fees, graduated according to horse-power, carrying capacity, etc., and the estimated damage of the vehicle to the road. In Louisiana, during the current year, these licenses will aggregate about $2 000 000, which constitutes a substantial part of the State fund for road work. It thus appears that, as far as concerns motor vehicles, which constitute the great bulk of highway traffic, the use of the public roads is not free by any means.

The rivers differ materially from the highways. The river is natural, the road artificial; the river was made by the Creator, and has existed for all time; the road is the result of man's efforts, and in most cases is as ephemeral as his other works; the river belongs to the Federal Government, the road to the State and local governments. Most rivers, however, require heavy expenditures to adapt them to the needs of modern navigation, and when improved by the Government they are, *pro tanto*, on a parity with highways. If it is fair, as the

speaker believes, to tax the truck for its use of the highway, why not tax the boat for its use of the improved river? There is no difference in principle between the two cases.

Boats Should Pay Licenses.—As far as the speaker knows, this question has not been raised in Congress. It should be, and steps should be taken to require license fees from boats on improved rivers and canals on a fair and equitable basis. Such fees would not be a more serious detriment to boats than licenses are to automobiles and trucks, and the adoption of such a license system would go a long way toward removing railroad opposition to waterways. Moreover, it would increase interest in the use of rivers, for men much prefer things that cost money to those that are free.

Great Importance of Highways and Trucks.—Thus far, the speaker has said very little about highways, which are also in the Transportation Trinity—Road, Rail, River. Owing to the development and general use of motor vehicles during the past 15 years, highways have assumed a position of commanding importance. The whole fabric of transportation has been greatly modified by the introduction of motor trucks. They carry a great volume of freight for distances of 10 to 150 miles; they are rapidly driving the short-line railway of 100 miles or less out of existence; they are ideal distributors of freight in less than carload lots in large cities, and to consignees within 75 miles of rail or water terminals; they relieved the situation materially during the fierce freight congestion of the World War, and carried great quantities of commerce several hundred miles in many instances; in the autumn of 1922 they handled 73% of the total net tonnage of manufactured products of New England; and they have demonstrated their usefulness so effectively that railroad leaders are urging that they supplement, and not compete with, railways.

Motor Trucks Essential to River Boats.—No development of the last half century has been so important to river transportation as the motor truck. During the long period of warfare between rail and river the capacity of boats to serve was limited to a narrow margin on each bank of the stream where horse-drawn wagons, carrying small loads, could co-operate with them. Now they can be served by powerful trucks carrying 2 to 7 tons, which can handle freight cheaply and rapidly to and from boats anywhere within 75 miles of the river, where highways are good.

Along the Mississippi, from New Orleans to St. Louis, this means an area of 120 000 sq. miles in the very heart of the valley, and a like area along the Ohio from Cairo to Pittsburgh, with its wonderful manufacturing interests. It means that adjoining these two streams, between the points named, there is an area of 240 000 sq. miles—five times as large as Louisiana—with more than 10 000 000 people, whose transportation needs can be supplied by boats and trucks when highways are perfected and river projects completed. Road building and channel work in these sections are being pushed vigorously. Friends of boats and trucks expect the most cordial co-operation between them, to the mutual advantage of both systems.

Appeal for Cordial Co-operation Between All Transportation Agencies.—In conclusion, the speaker again urges genuine friendship between the transportation agencies of the Mississippi Valley. If they pull together as brothers—all

for one and one for all—the development of valley commerce will be remarkable. Vast quantities of products from Atlantic and Pacific ports destined for this great interior basin will come by sea to New Orleans, thence by combined river and rail, or river and truck, to its consignees, the carriers returning with many commodities of the valley for exchange with their sea-coast brethren. A colossal volume of exports to foreign lands originates in Valley States, and can reach the sea *via* the Mississippi, with much saving in freight costs. The imports of these States are proportionately large, and just as their exports can save materially by following the down-hill routes to the Gulf, instead of crossing mountains to reach Atlantic or Pacific Coasts, so can like saving be made in transporting the products of foreign countries into the interior.

This cordial co-operation will result in such great increase in the commerce of the valley that the three systems—road, rail, and river—will have all they can do, and will reap good profits for their labors, while the Nation smiles on them, and enjoys the prosperity to which they have contributed so much.

DISCUSSION ON THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI*

By Messrs. Lewis Muhlenberg Haupt, C. E. Grunsky, and William T. Lyle.

Lewis Muhlenberg Haupt,† M. Am. Soc. C. E. (by letter).‡—In his excellent résumé of the present status of jetty location,§ Mr. Ripley has given seven types of construction based on form and position, which are functions of the physical localities, thus leaving the question of their adaptability open for further discussion.

Volumes have been written and great differences of opinion recorded as to the proper remedy for specific localities. In 1885, it was officially reported that:

"The methods are two (1) by dredging alone; (2) by using tidal scour between jetties, aided if necessary, by dredging. As to the first method, it has already been tried unsuccessfully. * * * The jetties should be so placed as to secure the greatest tidal scour practicable without seriously injuring the interior harbor, and without greatly endangering the safety of the jetties against undermining or of the Island from overflow in great storms."

Thus, the problem was reduced to the use of jetties in pairs with due consideration for the limitations as to the injurious results which might follow.

The precarious nature of the problem is well stated by Captain Hartman Bache, of the United States Topographical Engineers, in 1820, in his report on the opening of Roanoke Inlet, as follows:

"Whenever the course of Nature, in her marine operations, is to be governed, there is probably no subject within the range of the sciences, where so much is deduced from hypothesis, and where, necessarily, in the result there is so little certainty."‖

Since that time great progress has been made in meeting the demands of modern commerce; it remains to compare the results, and to determine the most economical and efficient forms of structure for creating permanent channels.

In 1869, when the writer was stationed in Texas, as U. S. Engineer in charge, his attention was directed to the delays of lightering over the bars which varied in depth from a maximum of 12 ft. to as low as 3 ft. Before a remedy could be applied successfully, it was necessary to determine what had caused the obstruction of the breaches along the coast. This led to the conclusion that, in purely tidal inlets, there is a resultant littoral drift which causes an advance of the island on the one side and a recession on the other,

* Continued from November, 1923, *Proceedings*.

† Cynwyd, Pa.

‡ Received by the Secretary, September 10, 1923.

§ "The Economic Location of Jetties", by Henry Clay Ripley, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1923, p. 1141.

‖ Armroyd's "Internal Navigation of the United States", 1830, p. 409.

leaving a deep gorge between and an outer and inner bar, as the tides sweep through. It was concluded that if this resultant could be controlled by a barrier which, while admitting the full volume of the tides to the bay, would at the same time protect the effluent ebb currents from spreading over the entire outer sector and also prevent the drift from entering the bar channel, the forces could be aided materially in securing and maintaining an adequate channel, by a fraction of one jetty.

In short, this remedy is well illustrated by the protective barriers erected to maintain an open roadway or railroad from snow, where the barriers are placed on the windward side instead of to the leeward as is so frequently done in the construction of jetties,* where the leeward one is built first, thus forcing the deposits seaward and adding greatly to the cost. This is illustrated by Fig. 46.

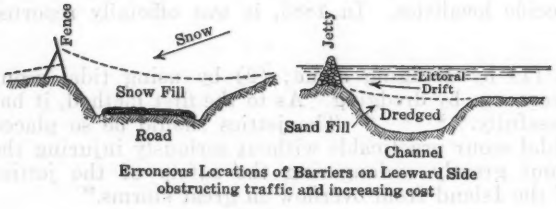

FIG. 46.

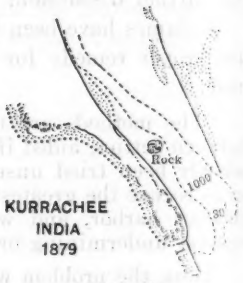

FIG. 47.

A striking instance of the efficiency of such control of the littoral drift is recorded in the effort to save the light-house at Barnegat Inlet (Fig. 48), where a single hooked jetty of piles was built by the municipality of Barnegat, 100 ft. west of the Government reservation, to check the flood drift, at a time when the sea was only 27 ft. from the base of the tower, and the wall and oil-house already had been destroyed, as well as the brush and stone jetty, 430 ft. long, built the previous year at a cost of $46 000. The pile jetty is 150 ft. long, cost $2 500, and is still in service. So immediate was the action that it was said to be "miraculous", and the later reports state that after more than two years, it also has caused a shifting of the channel, so that the water on the bar has been automatically deepened and the entire beach protected. The east jetty, which was also making beach, was not completed and, in consequence, has been partly destroyed, but no repairs have been necessary on the pile jetty, which has demonstrated its efficiency in both beach and channel improvements at a minimum cost.

These facts point strongly to the importance of controlling the littoral drift and defending the channel from its encroachments. This also has been done on the Gulf of Mexico, in one case described later.

* For a more extended description of the agencies available, reference is made to "The Physical Phenomena of Harbor Entrances", published by the Am. Philosophical Soc. of Philadelphia, December, 1887, and to the paper by Lieut. C. H. Davis, U. S. N., on "The Law of Deposit of the Flood Tide, Its Dynamical Action and Office", in the *Smithsonian Contributions*, Vol. III, December, 1851, endorsed by Professors Agassiz, Guyot, and Joseph Henry. See, also, *Transactions*, Am. Soc. C. E., Vol. LIV (1905), Part A.

As to the several types of jetties mentioned by Mr. Ripley, it is worthy of note that L. F. Vernon-Harcourt stated* that, in 1885, the fundamental requirement was the free admission of the tides, because "to diminish the tidal capacity within a harbour was the worst thing that could possibly be done." He was then describing the bar at the mouth of the Liffey, referred to as one

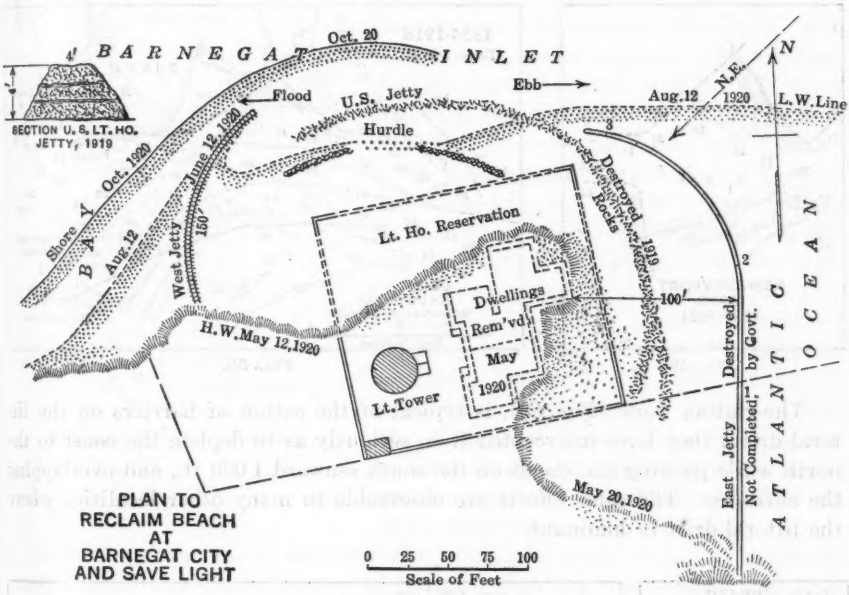

FIG. 48.

type of jetty by Mr. Ripley, who also refers to other instances in connection with natural promontories. An example of the latter type may be found at Kurrachee, India (Fig. 47), where an east pier more than 6 500 ft. long was built, about 1879, supplemented by a groin extending 1 500 ft. on the west, so that there were practically two jetties, and where the "removal of the bar has been assisted by dredging". A similar condition exists at San Diego, Calif., where Point Loma, about 10 miles long, separates the Bay from the Pacific, where a rubble mound jetty, 7 500 ft. long, has been built and where the original depths of 22 ft. have been increased by dredging to 26 ft., and now to 35 ft., for the largest vessels afloat.

The case of the Liffey, however, was peculiar, as the wall on the south, built in 1796, was isolated, so that a north wall was built (1820-1825), converging toward it, but leaving an opening of 1 000 ft. at the sea end and a breach of 600 ft. at its inner end to admit the tides, in order that the increase of the ebb, aided by dredging, might create the channel. The result was an increase of 7 ft. in 30 years, and it was said that "there was no other example of construction, for scouring purposes, which had proven so successful". It is, therefore, a type of convergent jetties supplemented by dredging, of which there are many precedents, a few only of which can be submitted, such as those at New-

* "Harbours and Docks."

buryport, Mass. (Fig. 49), the Columbia Bar, Washington-Oregon (Fig. 50), Rio Grande do Sul, Brazil (Fig. 51), the Sulina Mouth of the Danube (Fig. 52), and Wilmington, Calif. (Fig. 53), all of which have caused the bars to advance requiring jetty extensions, supplemented by dredging.

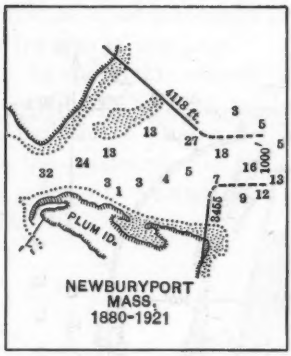

FIG. 49.

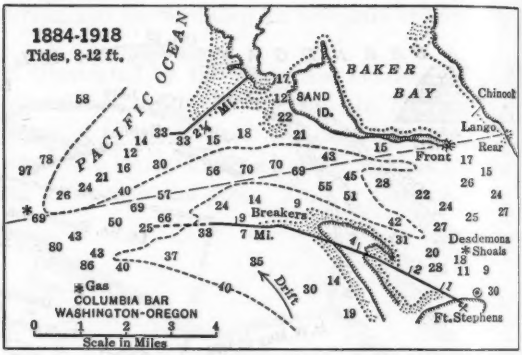

FIG. 50.

The Sulina work (Fig. 52), is typical of the action of barriers on the littoral drift: they have intercepted it so seriously as to deplete the coast to the north while pushing the shoals on the south seaward 1 000 ft., and overlapping the entrance. The same effects are observable in many other localities where the littoral drift is dominant.

FIG. 51.

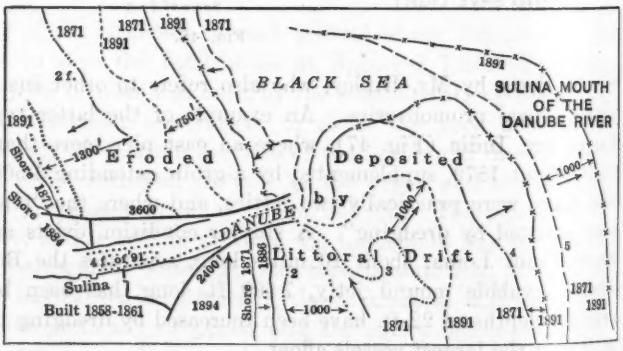

FIG. 52.

The third class of divergent or trumpet-shaped outlets is illustrated by the artificial canal cut at the mouth of the Maas (Fig. 54), where the channel was cut through by dredging, and a third dike became necessary (as at the South Pass of the Mississippi), because the width was too great, and checked the effluent, causing shoaling of the bar. An additional spur has been added to the south jetty to protect it from local scour.

In the fourth type, reference is made by Mr. Ripley to the opening of the harbor at Swinemünde by a single curved jetty supplemented by another nearly parallel, in order to arrest the eastwardly moving drift. As this is a mooted

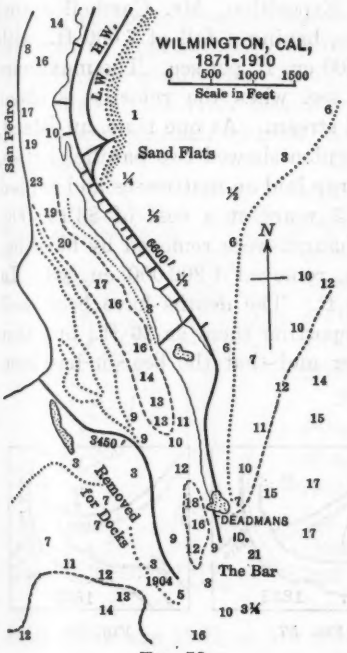

FIG. 53.

question, a few reduced copies of authentic surveys (Figs. 55, 56, 57, and 58) may serve to reveal the action of the forces since 1739, with the supplemental plans of the Baltic littoral (Figs. 59 and 60), showing that the outer spits have traveled eastward with their outlets, many miles from their roots, thus placing the curved jetty on the far or driftward side of the channel. This jetty is protected, however, by the shorter barriers on the west, placed to arrest the drift, which they have done, but still it is projected far beyond their extremities, requiring dredging and further extensions, while the reaction of the east curve aids largely in maintaining the navigation. The diagrams tell the story better than words. Without the protecting dikes on the west flank, the channel would have required constant dredging. Even at the South Pass, dredging was required to open and maintain the channel and the west jetty was almost buried by the débris from the curve on the east.

As to the sixth type of twin straight jetties, the record is full of instances, but it is generally the same story of constant shoaling, bar advance, and jetty extensions, even to the extent of double jetties with spurs inside and the bar still advancing, with inadequate depths.

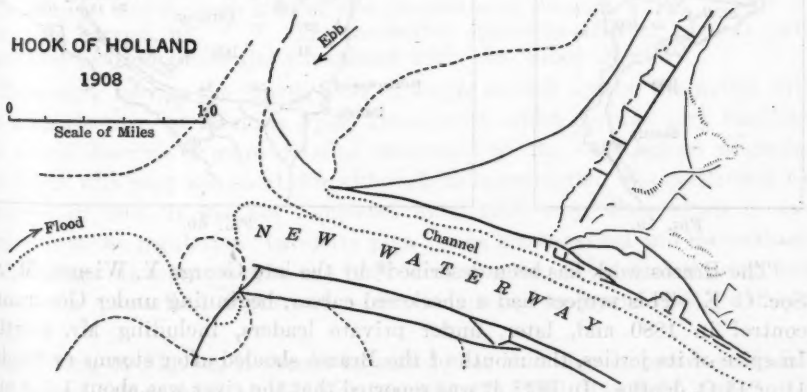

FIG. 54.

In this connection, it is worthy of note that, under almost similar physical conditions, the two types of parallel, straight jetties on the Gulf coast have given such diverse results as are shown at the mouths of the Brazos and the Panuca Rivers, both of which were reported on by the late Elmer L. Corthell, Past-President, Am. Soc. C. E.*

* "The Tampico Harbor Works: A Monograph", 1904.

In his brochure for the Pan-American Exposition, Mr. Corthell stated that the Panuca drains about 36 400 sq. miles, having a fall of 7 000 ft., with steep slopes and an average discharge of 190 000 cu. ft. per sec. The maximum flood discharge is about 215 000 cu. ft. per sec. when the velocity is about 7 ft. per sec., the river discharging in a solid stream. At one time, an island, bare at low water, blocked the entrance. The plan showed two parallel jetties, 6 500 and 6 800 ft. long, 1 000 ft. apart, of rip-rap laid on mattresses and capped with heavy blocks. They were finished in 3 years at a cost of $3 158 000. Cemented material and coral found in the channel were removed by blasting, and the flood of 1893, in less than 3 weeks, removed 1 202 000 cu. yd. In 1903, the channel had a least depth of 27.8 ft. The depths have been well maintained, the *United States Coast Pilot* reporting them as 26 ft., but that the outer ends of the jetties are under water and that the beacon has been destroyed.

FIG. 55.

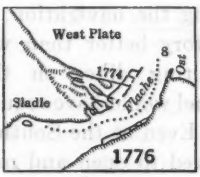

FIG. 56.

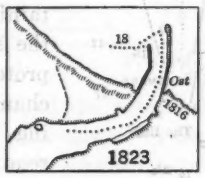

FIG. 57.

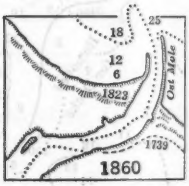

FIG. 58.

FIG. 59.

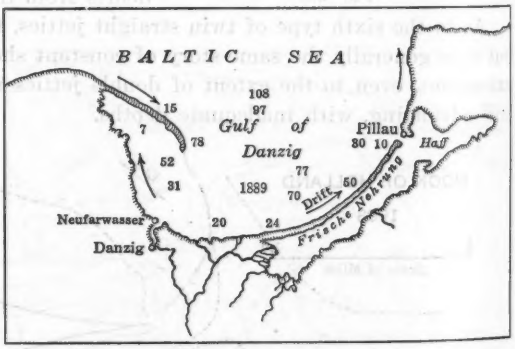

FIG. 60.

The Brazos work has been described* by the late George Y. Wisner, M. Am. Soc. C. E. This project had a checkered career, beginning under Government control in 1880 and, later, under private leaders, including Mr. Corthell. In spite of its jetties, the mouth of the Brazos shoaled after storms or floods to 11 or 13 ft. depths. In 1922, it was reported that the river was about 1 300 miles long and had a drainage basin of 41 700 sq. miles. It discharges from 245 to 66 000 cu. ft. per sec. and, at the highest floods, "probably 80 000, reaching a highest of probably over 100 000 sec-ft., not recorded". During the heavy June flood, the bar had shoaled to 13½ ft., requiring frequent dredging. Therefore, it would appear that notwithstanding the "frequent and heavy rises of the

* "The Brazos River Harbor Improvement", *Transactions*, Am. Soc. C. E., Vol. XXV (1891), p. 519.

Brazos River that have occurred since * * * 1918", the discharge of this basin is unable to create or maintain depths of much more than 13 ft., as compared with approximately 30 ft. at the Panuca, under apparently somewhat less favorable conditions. The total expenditures by the Government for the lower part of the river was about $1 358 000. In both cases, the jetties are straight and parallel, with no interior tidal reservoirs.

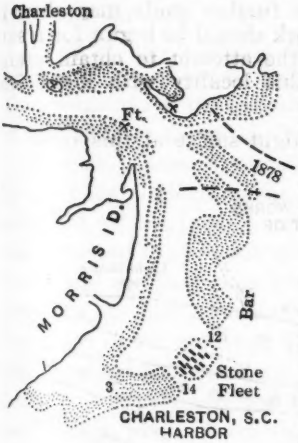

FIG. 61.

The jetties at the Brazos are 600 ft. apart, with interior spurs at intervals, while those at the Panuca are 1 000 ft. apart, with no spurs. The question naturally arises as to "the proper distance apart which will maintain the required depth without dredging, under such variable conditions of discharge." This problem does not arise with one jetty.

The seventh type tells its own story. However, there is a sequel which is vital in the solution of all these cases relating to the influence of the resultant forces in barricading ocean bars. During the Civil War, it was deemed necessary to close the entrance to Charleston Harbor (Fig. 61), and for this purpose a fleet of twelve vessels, loaded with rock, was sunk on the bar over which there was 12 ft. at low water. In 1888, the Superintendent of the U. S. Coast Survey, the late J. E. Hilgard, M. Am. Soc. C. E., cited the sequel when he stated that:

"The vessels were placed checkerwise, in such manner as to impede navigation, while interfering least with the discharge of water. The effect, nevertheless, was the formation of a shoal in a short time, and the scouring out of two channels, one on each side of the obstructions, through which 12 and 14 ft. can be carried. * * * The successful operation of your plan is well illustrated by this accidental experience with the 'Stone Fleet'."

Returning now to the fourth class of single curved jetties, the writer will cite a single case, at Aransas Pass, Texas, with which he was very familiar and which deserves a more detailed treatment as the only known example. The life of this jetty was short, for although its construction was authorized by Congress in 1902, it was not completed until 1906, soon after which it was merged into the regulation "two-jetty plan" to be accompanied and maintained by dredging. However before the later work was let, the detached jetty had secured the prescribed depth and width at a very moderate cost, "without dredging".

Referring to Fig. 62, it should be stated that this map embodies four distinct efforts which need elucidation, namely, the "Old Mansfield Jetty", the Nelson Jetty, soon destroyed; the Reaction or "North Jetty"; and the extension of this last to the shore in connection with the South Jetty, all of which have an instructive and unique history. This inlet had been under consideration since about 1853, and the requisite depth of 20 ft. was not secured until 1907, "without dredging".

The peculiar physical difficulties are stated briefly in the report of the Board of U. S. Engineers, of 1887, as follows:

"The problem * * * is by no means an easy one. Briefly, they [the difficulties] are, want of stability, instability of foundations, shifting sands, the presence of the sea-worm, the long haul for stone, and discomforts of a tropical climate.

"There is good prospect of success by the use of two jetties * * * the location is approved, subject to such changes as further study may show to be expedient. No money is now available. No work should be begun for a sum less than $500 000, as no policy is worse than the attempt to obtain useful results under such circumstances as exist at this locality with insufficient means."

One member, however, believed that one jetty might suffice at this inlet.

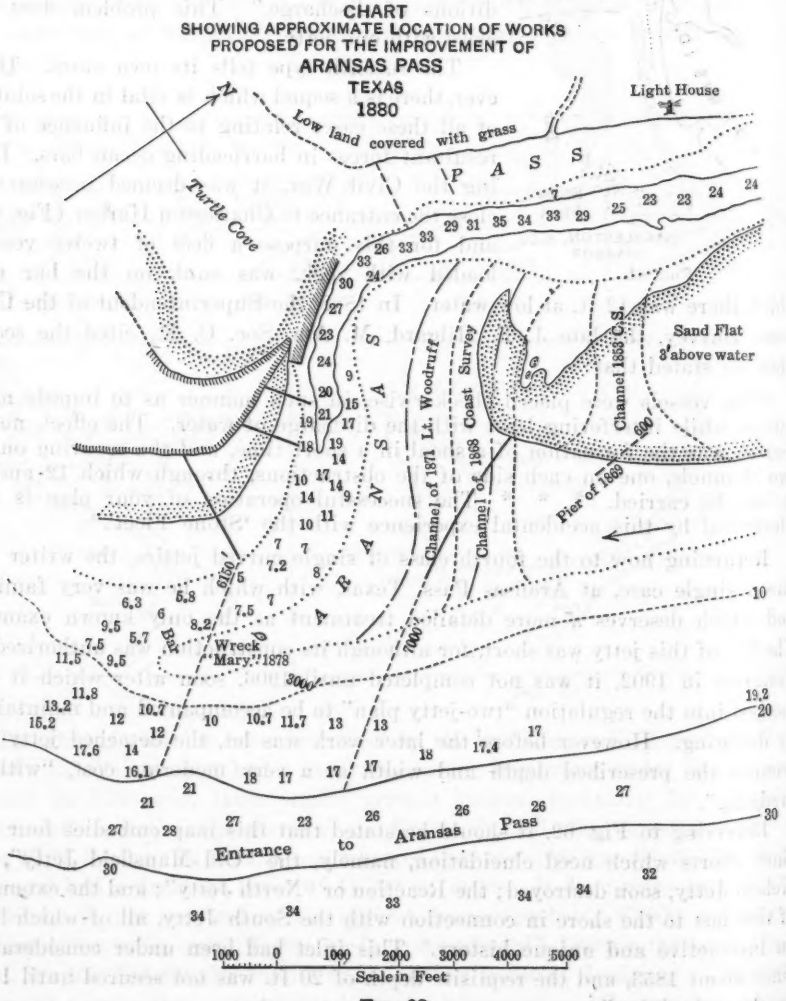

FIG. 62.

At this date, the Mansfield Jetty which was begun in 1880, and was 5 500 ft. long, built on the far, or south, side was abandoned, with "insignificant results." The channel had crossed the jetty. It was recommended, therefore, that two jetties be built at a cost of about $2 000 000. The construction was started and the outer end was curved toward the channel, but all "work was suspended in May, 1889."

Prior to this a project to secure a 12-ft. channel by means of two jetties 3 000 ft. apart, at a cost of about $760 000, had been approved, but not executed. The conditions of the bar with its depth of 6 ft. and the progressive movements of the inlet are shown on the plans of 1880 (Fig. 62).

In 1890, a private company was chartered by the State of Texas, as well as by the Federal Government, to secure a 20-ft. channel. It built a jetty on the south side of the inlet, composed of wooden caissons filled with rock and sand, which pushed the bar out with a resulting depth of 8.5 ft., before it suspended. Later, the company was re-organized for the construction of the "Reaction jetty" on the north flank of the bar, on which work was begun in 1895, to be completed in ten months. The work was started on the outer slope of the bar, and extended shoreward on curves of variable radii, compound and reversed, but it was soon discovered that the remains of the rock-covered jetty of 1888 were still in place, obstructing the tidal scour. This led to a suspension of the effort and a return of the franchises to the Government on the condition that the work should be completed on this design, after the removal of the part of the old jetty across the channel. This was not done until 1904, and a part of the jetty (Fig. 62), was not completed until 1906. In December, 1907, it was reported on good authority that the predicted channel had been obtained "without dredging". These results are shown on Figs. 63 and 64 which are copies of U. S. Coast Survey charts, and indicate both the previous and the subsequent depths on the bar. The latter are in fathoms which, reduced, give 28.5 and 25.5 ft., respectively, where before they were 6 and 8 ft. The ruling depth of 17 ft. soon after disappeared. It must be recorded, also, that, desiring to secure still greater depths, the Government immediately recommended a return to the "two-jetty plan", to be supplemented by dredging, and, for this purpose, closed the tidal entrance at the shore end of the north jetty, on February 27, 1909. A survey made in March, 1909, showed that the channel had shoaled to 13.8 ft. at its outer end where it had previously scoured to from 28 to 30 ft. deep. It gradually recovered its normal depths, however, and has secured a channel of nearly 25 ft. throughout, thus rendering unnecessary the proposed extension of both jetties to be supplemented by dredging at great expense ($2 325 000). It appears, therefore, that the single curved jetty on the windward side was in existence only two years, from July, 1906, until July, 1908, when the closure began, and that it required nearly three years more before the required channel was restored, "without dredging".

The several phases of the changes caused by these operations can be appreciated by studying the official charts of the U. S. Coast Survey, Figs. 65, 66, 67 and 68, with the accompanying notes, from which it appears that the old Nelson Jetty was not a factor in the results, as it was destroyed soon after it was built; that the Mansfield barrier which was reported as "not acting as an

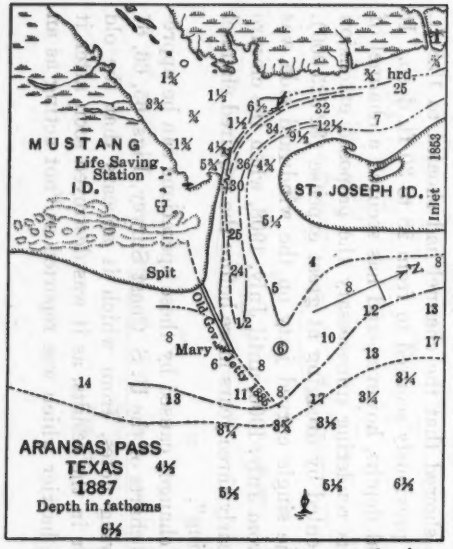

Fig. 63.—Old Government Jetty on the leeward side abandoned with "insignificant results." Cost, $550 000. (6 to 8 ft. on bar. In harbor, 32 to 35 ft.) The channel was reported to have crossed the Jetty, which had "disappeared" and was "not acting as an obstruction." It was later found to be in place, covered with rock at from 8 to 12 ft. depths and was only partly blasted out in 1904, leaving fragments at 16½ ft. depth as late as 1911.

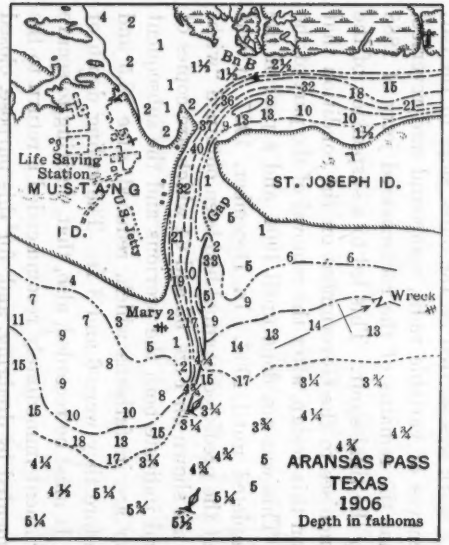

Fig. 64.—Jetty, reported as completed after 10 years of delay. Channel forming 28½ to 25½ ft., with ruling depth of 17 ft. The cost to the Government was less than $500 000. In December, 1907, it was reported that there was a 20-ft. depth entirely across the bar, but no survey was made and no additional work was done for some time. When the jetty was connected with the shore, it caused the channel to shoal to 13 ft. in less than a month, and it required more than two years to recover the 20 ft., by the reduced flow of the tides "without dredging." The protrusion of Mustang Island canalized the channel, automatically. In 1906, it was recorded that "a rock lump with 16½ ft. of water was found in the channel, believed to be a portion of the old Government Jetty overlooked by the inspectors."

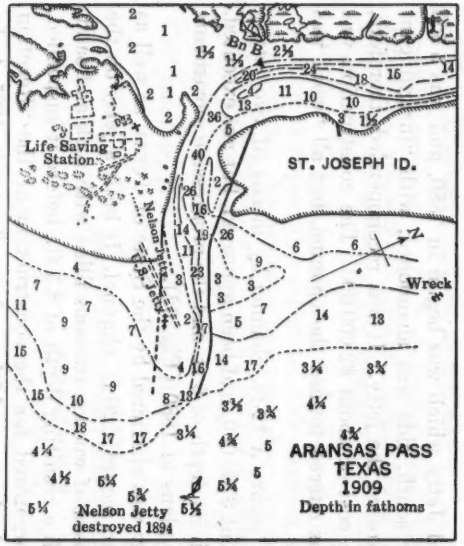

Fig. 65.—The tidal entrance at the inner end was closed February 27, 1909, and a "South" Jetty was started to convert it into the "two Jetty plan," resulting in a shoaling from 28 to 13.8 ft. in less than a month, at the outer end, and also in losses in the harbor of about 8 ft. It was not until 1911, that the enfeebled tides recovered the 20 ft. channel without dredging, and this notwithstanding the severe storms of 1908, which caused a shoaling to 16 ft. due to the erosion of about 60 acres from Mustang Island, and would have closed the inlet but for this reaction jetty. The "Nelson Jetty" had no existence at this date (see Fig. 64, 1906). It was built in 1892 and soon after largely destroyed.

of water was found in the channel, believed to be a portion of the old Government Jetty overlooked by the inspectors."

destroyed.

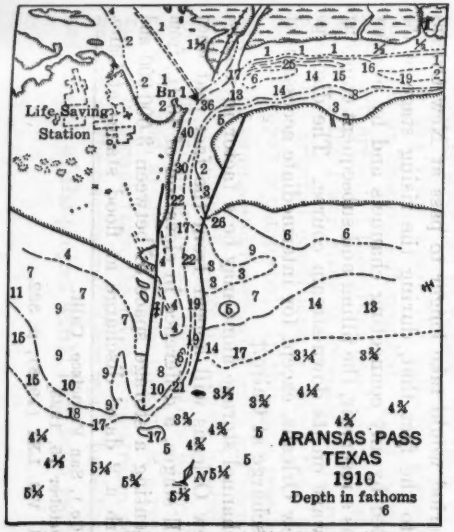

Fig. 66.—This survey indicates an increase in depth of 8 ft. at the outer end during the year and a general improvement in the channel along the reaction breakwater, the ruling depth being 17 ft. at low water, but the harbor is shoaling rapidly. The South Jetty has been extended beyond the north, with a middle ground shoal of 4 to 6 ft., thus requiring the tides to enter through a congested channel more than 1.6 miles in length, having a minimum sectional area less than that at the inlet, thus reducing the efficiency of the ebb for scouring. The "reaction" of the concave jetty has, however, restored and maintained the navigable channel along its trace. (See Figs. 67 and 68.)

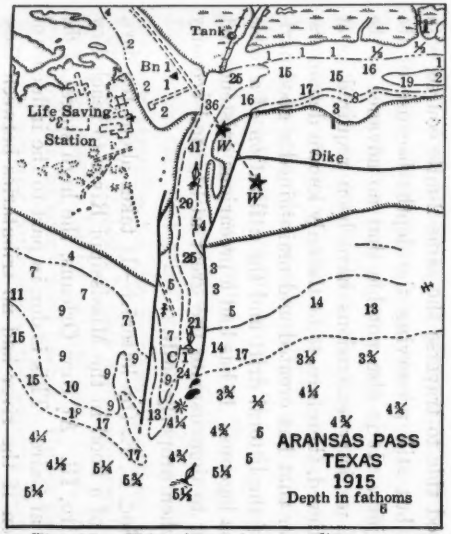

Fig. 67.—This phase shows a disappearance of the Nelson Jetty ruins, and a removal of a portion of the old U. S. Jetty and a deepening of the Harbor by dredging. The ruling low-water depth is 21 ft. abreast of the Old Jetty, of which 1 070 ft. had been sunk by dynamiting to a depth below 25 to 30 ft. The 20-ft. depth was recovered in 1911, and in 1912 it was reported, "There is a gain of 150 ft. in the width of the 20 ft. channel." This was due to the removal of more of the old sill across the channel, as had been predicted in the report of 1911. In none of these surveys is there any indication of an advance of the bar beyond the end of the breakwater. In December, 1912, it was proposed to extend both jetties, at an estimate of $2 325 000, to secure a 25-ft. channel, but this has not been found necessary. (See Fig. 68.)

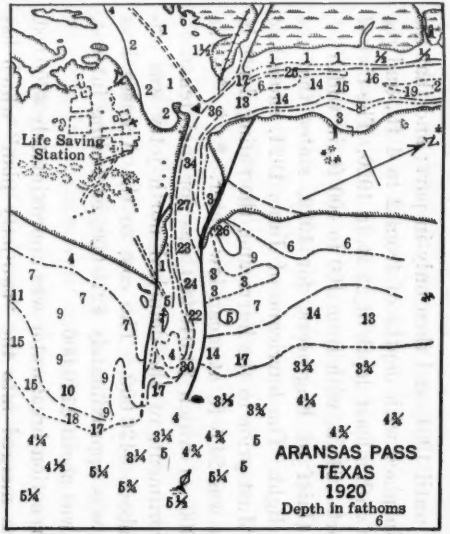

Fig. 68.—U. S. Coast Survey Chart of 1920, showing the stability and increased depths after the storm of 1919, "the hurricane of 1921, resulting in no damage to the improvement." Prior to the hurricane the ruling depth between the jetties was 24.5 ft.; least width, 60 ft. The hurricane caused a shoaling at one spot to 22.6 ft. near the site of the old jetty. "This port has been placed on a parity with Galveston as regards freight rates." It is proposed "to construct two experimental spur-dikes on the north jetty at an expenditure of about $80 000." (p. 1094, Report, U. S. Engineers, 1921.) The Survey of January, 1917, indicates depths of from 25 to more than 30 ft. between the jetties, and 27 ft. beyond the end.

obstruction", was not removed until 1904, and then only in part; and that the middle-ground shoal remains in place with depths of from 2 to 4 ft., and the channel hugs the reaction jetty with a least depth of 22 ft. in 1920. The *United States Coast Pilot* to date reports "24 ft., with from 100 to 400 ft. in width".

The report of the U. S. Chief of Engineers, for 1922, states that the damage from recent storms is light. The hurricane of June, 1921, caused no damage to the improvement. "Just prior to the hurricane, the ruling depth in the channel between the jetties was 24.5 ft. at mean low water. * * * The total expenditure was $3 160 000, of which $1 040 000 was for maintenance".

The total cost to the Government, of the detached north jetty was officially reported to have been about $436 000. The cost to the Aransas Pass Harbor Company for its work was approximately $270 000, excluding interest, and that for the Nelson Jetty was about $110 000.

Thus, the total cost of the reaction breakwater was approximately $700 000. Its identity, however, has been merged into the "two-jetty plan" which has retarded the inflow of the feeble tide (about 14 in.), and shoaled the harbor, requiring both the ebb and flood tide to traverse the same 2-mile course, with reduced velocity and volume, but still preserving the depths because of the reaction of the curved north jetty. It is also notable that no advance of the bar has occurred and, hence, no jetty extensions have been required as at many other localities. It is believed, therefore, to be the only known demonstration of a single offshore barrier that has created and maintained a good navigable channel by the control of the littoral drift and the utilization of a feeble tide flowing into almost tideless lagoons. It has had a precarious existence, but has been of sufficient duration to demonstrate the great utility and economy of the single offshore barrier when properly built.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).†—It takes about ten days, or a little longer, for the crest of a flood in the Mississippi River to reach New Orleans, La., after passing Cairo, Ill. At New Orleans, the high stages of the river are more protracted than at Cairo. This is due in part to the influence of the river's tributaries; but even if the river had no tributaries between these two points and all the water were confined between the levees, the water which passes Cairo during a flood period would take longer to pass at New Orleans than at Cairo. This is due to the fact that, during the rising stage of the river, its channel, supplemented by any connecting channels and by overflow areas between the levees, acts as a reservoir, the filling and subsequent emptying of which elongates the flood wave on its down-stream course. The discharge at the crest of the flood at New Orleans, except for intermediate accessions of water, is less than the peak discharge at Cairo.‡

To what extent the river channel storage alone may be a factor in modifying the peak discharge at New Orleans will appear from the fact that when the river is almost at bank-full stages, a difference in elevation of 10 ft., from Cairo to New Orleans, representing a storage space of between 3 000 000 and 4 000 000 acre-ft., would absorb a 2 days' discharge at flood stages at Cairo.

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† Received by the Secretary, October 15, 1923.

‡ *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 332.

The problem, however, is not simply a problem of determining how a flood wave would be modified in shape in passing down the river; it is complicated by the fact that the discharge of its tributaries is added to that of the river at flood stage at Cairo. As the possible discharges from these tributaries are also fairly well known, as well as the outflow at flood stages through such outlets as that to the Atchafalaya River, a fair basis exists, for the attack on this problem with the aid of mass curves. With these data assembled, it should be possible to determine the elements of mass curves representing the various possible accessions of water from all sources to the main stretches of the river above New Orleans and, with proper allowance for reservoir effect and the time required for the crest of the flood wave to travel from point to point on the river, also, to determine the shape of the mass curves of discharge which, under various assumptions as to the relative periods covered by the high water in the main stream and in its tributaries, would obtain at New Orleans if all the water were confined to the river, or, as an alternative, which would obtain if some of the water was allowed to escape at selected points. By this method of procedure, the question as to whether or not the flood volume to be passed at New Orleans in a given time, will or will not overtax the channel capacity at the highest allowable river stage, can be removed, in all probability, from the realm of mere conjecture. If it thus can be shown that, at the highest allowable stage, the river cannot discharge the volume of water arriving from above, then there is need for relief outlets. If such relief outlets are not provided at selected points so that the flood water can escape from the river under control, the river will find a weak spot, as it did in April, 1922, and will make its own relief outlet. There is no alternative.

In the case of some rivers, as, for example, the Sacramento River in California, the fact that relief outlets are imperative, is easily recognized. There are long stretches on the Sacramento where the capacity of the river with levees about 20 ft. high is only about one-eighth of the required capacity at a high flood stage. Under natural conditions, the surplus water went over the bank into the adjacent flood basins. Under the adopted scheme of flood control, now nearing completion, the surplus water will be allowed to escape at selected points over weirs and instead of spreading out in the flood basins will be confined to so-called by-passes. In these by-passes, it will flow to a lower point on the river where the capacity, already large, can be and is being increased by channel enlargement. It is interesting to note that, as a result of the withdrawal of the river's flood basins from inundation, the maximum discharge of the lower river is expected to increase from about 250 000 sec-ft. under the original conditions to more than 500 000 sec-ft. when the flood-control project is completed. In this case, possibly, the attempt to reclaim completely the swamp and overflowed lands has been carried too far. It would have been better to allow the large flood basin located between the Sacramento River and its principal tributary, the Feather River, to remain open to the river at its lower end, at the same time making it a recipient of flood waters that are allowed to escape from the river at up-stream points. The whole flood-control problem of the Sacramento River would have been materially simplified thereby and the cost of the project would have been reduced.

Turning again to the Mississippi River, the question naturally arises as to whether it is wise to withdraw all its flood basins completely from its overflow area. This should appear from an analysis of the problem as previously indicated. It is not enough to be able to point to the wonderful success of the Mississippi River Commission's project, as evidenced by the constantly decreasing number of annual levee breaks and the decreasing area of lands submerged at the flood stages of the river. The door should be left open for the next step which may be, as many believe, in the direction of allowing some of the water to escape under control through relief outlets.

WILLIAM T. LYLE*, ASSOC. M. AM. SOC. C. E. (by letter)†.—If there was a National university in the United States, the National Hydraulic Laboratory would constitute very properly a part of its engineering equipment. As far as such equipment in general is concerned, much might be said in advocacy of a National university. As matters now stand, each engineering school has its own laboratory equipment, a part of which is used for routine instruction and a part for research work. Duplication of apparatus could be reduced if research work was done at a National university where it would be facilitated by larger and more extensive equipment. This, however, is not the subject under discussion. Mr. Freeman thinks the hydraulic laboratory is necessary in the interest of hydraulic science in general; but, in particular, in order to render more effective the Government appropriations for river control‡. He has placed the emphasis on the second desideratum, and the writer is convinced that he has clearly established his contention.

In the "river flume", important results will almost certainly be reached. Homology between the tests and the phenomena in the Mississippi River may be discovered; but even if a complete homology cannot be established, it is reasonably certain that important light will be thrown on the phenomena of erosion and silt deposition and on the proper design of spur-dikes, retards, mattresses, or other controlling structures found to be most advantageous. It is also reasonable to hope that information may be obtained concerning the debated question of the desirability of straightening a sinuous channel.

On the other side of the laboratory, in the "weir flume", valuable data undoubtedly will be obtained on the subjects of pipes and conduits, free and submerged weirs, dams, "Venturi flumes", loss of head due to enlargement of section, etc. The merit of the "weir flume" lies in its great size. The writer is convinced that in several large American laboratories too much attention has been given to the matter of head and not enough to that of quantity. In Mr. Freeman's laboratory, provision has been made for quantities up to 600 sec-ft., an amount equal to the dry-weather flow of a good sized river.

Some years ago, the writer suggested the building of one-story laboratories. Of course, such laboratories would be limited in their scope, as experiments on orifices under high heads could not be made. Such experiments, although valuable, are not of such practical worth as those on the flow over weirs and

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.
† Received by the Secretary, October 24, 1923.
‡ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1185.

dams and in conduits and canals. Power investigations for high-head conditions can be made only in laboratories especially designed for that purpose.

An ideal site for a university laboratory is beside a stream, the water supply being taken from behind a mill dam. The building should be of an elongated type. In it, experiments can be made on sewer and water pipes of a large range of sizes, on Venturi meters of different sizes and shapes, on open "Venturi flumes", on dams and drowned weirs, and on siphon spillways. A head as small as 12 ft. would be amply sufficient, even allowing for loss in the measuring tank, as phenomena in the tests mentioned do not depend on absolute head, but rather on a difference of head. For instance, in sewers, the loss of head is only a small fraction of 1 ft. per 100 ft.; and in a water pipe, under ordinary working conditions for all large sizes, the drop in the hydraulic gradient per 100 ft. will also be only a fraction of 1 ft. The laboratory should measure at least 200 or 300 ft. in length and need not be under one roof, a test house at each end connected by piezometer tubes for differential gauge connections being sufficient. Such a laboratory could be built at small cost, except, perhaps, for land and riparian rights. In it, a wide range of instructional experimentation could be carried on, and in lieu of other provision for graduate research work, as hinted at in the beginning of this discussion, investigations of great practical value to the profession could be made.

THE DISINTEGRATION OF CEMENT IN SEA WATER

Discussion*

By Messrs. Charles A. Newhall, Eduardo de Castro, and R. Feret.

Charles A. Newhall,† Esq. (by letter).‡—This able paper is the first authoritative recognition of a condition that long ago might have been foreseen and guarded against by the cement manufacturers. No one now can controvert the statement that under many conditions even the most carefully made concrete will disintegrate.

The way is cleared for those who are familiar with the manufacture and use of Portland cement, to set forth the conditions under which it is not safe to use this important construction material and, more important, to undertake the research necessary to determine the underlying causes of these failures. The opportunity is now presented to the manufacturer and the inventor to devise a type of cement that will endure under certain conditions that are recognized to be fatal to the permanence of Portland cement concrete.

During about twenty-two years of close association with the manufacture, testing, and use of cement, the writer has had opportunity to note many interesting matters which have a bearing on the questions raised in this paper. Several of these observations have been published;§ the most interesting is that of the rôle played by carbon dioxide gas dissolved in ground-waters. The destructive action of this gas which is always present as carbonic acid in normal waters, has been overlooked because it is not reported in the usual water analysis. It must be determined on a special sample and by special analytical methods. This acid is present in normal waters in quantities varying from a trace to more than 3 000 parts, by weight, per million (when cold and fully saturated water is in contact with the gas). Small concentrations seem to be the more destructive. Carbonic acid has been termed by soil chemists the "universal solvent", and its action in weathering rocks has been long recognized, but remarkably little experimental data are available on the reactions and concentrations involved; this is probably due to the difficulty of simulating soil conditions in the laboratory, and offers a very interesting field to the research chemist and the designing engineer. For example, an engineer, working on the design of a thin-walled, concrete structure, informed the writer that it would cost $200 000 for the extra inch of concrete needed to compensate for the dissolving action of carbonic acid on the outside of this structure during its estimated useful life of twenty-five years.

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* Discussion on the paper by William G. Atwood and A. A. Johnson, Members, Am. Soc. C. E., continued from November, 1923, *Proceedings*.

† Cons. Chemist, University Station, Seattle, Wash.

‡ Received by the Secretary, October 29, 1923.

§ "The Disintegration of Concrete in Railway Tunnel Linings," *Proceedings*, Pacific Northwest Eng. Soc., 1916; "Incrustation of Vancouver Sewer Pipe," by A. G. Dalzell, *Canadian Engineer*, November, 1918; and "Action of Ground-Waters on Cement Sewer Pipe," *Contract Record*, March, 1922.

Carbon dioxide is probably the main cause of the disintegration of concrete structures both in sea water and in ground-waters. In sea water, and in the so-called "alkali disintegrations", the trouble has been laid mainly to magnesium sulphate and, to a lesser degree, to the calcium and sodium sulphates. A double decomposition takes place whereby the nascent calcium hydroxide (which is set free in an extremely active form whenever the modern Portland cement hardens) re-acts with the sulphate radical to form calcium sulphate, and the hydroxyl radical combines with the magnesium and sodium to form the hydroxides of these two elements. (This is the reaction that gives the "furring" on concrete walls on which a white fluffy deposit of magnesia and soda forms when gravel and sand washed in salt water are used.)

Under ordinary conditions of moderate moisture, the only harm from these reactions would be the formation of a thin surface "fur" or, at most, a layer of soft scum possibly $\frac{1}{8}$ in. thick. As chemists term it, an equilibrium would be reached, and the reactions would stop.

However, if seepage water, or flowing ground-water, or a "restless sea" washes the concrete structure, the reactions noted will continue indefinitely and the structure will be destroyed, for the reason that equilibrium can never be reached. The carbon dioxide that is always present in all natural cold waters, continually dissolves and carries away the magnesia as bicarbonate, and the flowing water washes out the soda.

Further, it has been shown that relatively pure water containing only small concentrations of magnesium salts, when charged with carbon dioxide gas, will dissolve the calcium compounds out of the hydrated cement and eventually will cause the disintegration of the structure. The calcium is removed in the form of the relatively soluble bicarbonate.

It is unfortunate that investigators who have reported on concrete disintegration and resistant cements have not been familiar more often with the chemical and manufacturing phases of the subject. Relatively few realize the fundamental changes in composition and processes that have taken place in the manufacture of cement. The cement that is standard on the Pacific Coast to-day is a totally different product from that made twenty-two years ago. Theoretically, the modern cement should be much more likely to disintegrate under the action of ground-water and of sea water than the old-time cement.

In attacking the problem of producing a resistant cement, the investigators should always keep the practical aspects strongly in mind. It is a great joy to experiment and work with unusual chemicals and cements, but such work is costly.

This is a case where all branches of the Engineering Profession should co-operate on a practical basis and take lessons from each other. The civil engineer should set aside precedent and admit that an entirely new set of specifications will have to be made to cover different types of construction. Portland cement will be only one of several hydraulic binding materials. The research chemist, the analyst, and the chemical engineer should realize that the civil engineer is compelled to work with construction organizations the operations of which are highly standardized and largely routine. Therefore,

the new cements must compare in price, appearance, and working properties with the Portland cement of to-day.

The writer thinks geology holds the answer. Some of the earlier concrete enthusiasts stated that their products were as "permanent as the everlasting hills". A geologist knows that some "hills" are extremely unstable. He also knows and can point out the types of minerals that are most resistant to the action of the elements. The chemical engineer, with the help of the research chemist and the analyst, should be able to produce an artificial sandstone that "beats Nature". When hydrated, modern Portland cement is comparable with the unstable zeolite minerals, whereas a cement should be more like the stable plagioclase feldspars, or the resistant opal, or chalcedony.

From present indications there does not seem to be much hope from the aluminate cements, as the raw materials are limited in quantity and are costly. About fifteen years ago, the iron ore cements were esteemed highly for sea-water construction. In these cements, the alumina of the usual cement formula is replaced largely by iron oxide. At one time, the writer was compelled, through the scarcity of proper clays, to make an iron ore cement. The composition was easily controlled and many hundred barrels were made and sold to the regular trade. However, the product did not gain popularity on account of its black color and relatively slow-setting properties, although it hardened well and worked very "smooth". If research and tests in sea water and soil waters demonstrates that iron ore cement is resistant, as the early advocates claimed, it will not be difficult to manufacture this cement.

Experience and theory point most favorably to a judicious blend of the modern Portland cement with a pozzuolana-like silicate. This type of material, having a high content of colloidal silica, is easily obtained almost anywhere in the Western lava "barrens", where it is commonly called "ash" or "cinder". The tufa used in the construction of the Los Angeles Aqueduct was a material of this type. The results obtained there were in the main satisfactory, although much adverse comment was current at the time. ("Sand cement" as used at the Arrowrock Dam should not be confused with cement of the tufa or pozzuolana type.)

It is to be hoped that the authors and the Committee on Marine Piling Investigations will have the united support of the Engineering Profession to the end that this essential problem of developing a resistant hydraulic cement may be solved promptly and without the friction that unfortunately has developed heretofore whenever individuals have attacked the problem.

EDUARDO DE CASTRO,* ESQ. (by letter).†—Several phases of this interesting subject are worthy of note. The writer will discuss briefly those that have come under his observation.

Causes of Disintegration.—From some chemical researches made by the writer, it appears that disintegration is caused by the action of the sulphate of magnesium of the salt water on the free lime of the cement. Some doubts arise from the fact that the concrete is saturated with sea water, and it is

* Engr. of Roads, Canals, and Ports of Spain, Gijon, Spain.
† Received by the Secretary October 29, 1923.

difficult to determine whether the compounds that appear during the analysis belong to the water or to the disintegrated cement. In spite of this, however, it seems that lime and magnesium play the most important parts in decomposition. It is well known that disintegration is hastened if large quantities of lime are mixed with the cement, and, also, if the specimens are submerged in concentrated solutions of sulphate of magnesium. This opinion is confirmed by the careful experiments of Mr. Charles J. Potter who has made thousands of tests in concentrated solutions of every salt found in sea water.

Disintegration assumes different appearances. The writer has seen blocks stored on a beach (where they were daily covered by the tide) that presented the same cause of disintegration as that studied by Candlot, that is, the formation of sulpho-aluminate of lime. There were many eruptions over the surface, as if grains of gunpowder had been embedded in the mass. In most cases, these explosions do not show, but the whole mass is destroyed.

Some failures occur in a short time, and, in such cases, the decomposition probably begins within a few days after the concrete has been submerged in water. With other failures, there is a long delay between submersion and the first sign of decomposition. This last case is worthy of attention. Probably the free lime already has been converted into an inert carbonate. The hard burned lime also has combined with the carbonic acid. The writer has heated lime in an enamel kiln to the high temperature of 1 500° cent. (which is hardly reached in the cement kilns), and lime thus obtained was not hydrated and carbonated as quickly as that treated at a lower temperature, and yet not so slowly as to be the cause of disintegration that appears only after a long time. In such a case, it is possible that the lime is produced from some aluminates that are slowly disintegrated.

Many failures occur near the level of the low tide, but, to the writer's knowledge, blocks and concrete bags have been destroyed 12 m. below the lowest level of the tides.

As regards exposure, the influence of the mechanical action of waves is very evident. On the ocean side of a breakwater, concrete was found to be destroyed, but on the harbor side, although attacked, it conserved its form and was easily consolidated.

Free Lime.—Free lime, although not originally contained in the cement, is produced during the setting process. By the use of the microscope, the writer has confirmed this opinion of Le Châtelier and Michaëlis. It is useless, therefore, to neutralize the lime by aeration, or by a current of carbonic acid gas, or by the gas of the kiln, which easily could be done.

As previously stated, there is a third cause of free lime, namely, the disintegration of some unstable aluminates. Uncombined lime is easily carbonated; it also can form some combinations with the silica of the cement. Several quick-setting Spanish cements contain large quantities of lime. Although such cements are used in poor and permeable concretes, manipulated without any care, they form mortars that are not attacked by sea water. Such cements are mixtures of lime and a baked clay that has acquired pozzuolanic properties.

A lime that will combine with carbonic acid or with silica is not harmful, but rather the kind that remains free after hardening, without any possible combination. To the latter only must the term "free lime" be applied, but not to lime contained in cements before water has been added.

Silica.—It is necessary to consider only the silica that is capable of making chemical combinations with the lime. The hydraulic index of the cement does not change when some quantity of quartz is added to it. The inert silica that remains in the same state throughout the processes of setting and hardening, and the gelatinous silica that is unstable and not found in concretes or in pozzuolanas, are not important. Only silica soluble in alkalis is here important; good pozzuolanas must contain this element.

Causes of Firmness of Some Cements.—Most cements disintegrate in salt water, but not all. While the sulphate of magnesium is acting on the lime, another protective process is being set up by means of both the silica and the carbonic acid. Disintegration is a struggle, and the process that is successful, will determine the result; either sulphate of magnesium proves effective and failure occurs, or the action of carbonic acid and silica prevails, securing the preservation of the cement. It is most essential to assist this latter process.

Standard Cement.—Cements are tested by experiments not suitable for marine conditions. Specifications for structures in sea water are made more stringent than those for construction that has proved satisfactory in air, on the theory that, by the added precautions, structures will be produced worthy of acceptance for use in sea water.

High strength is indeed a good quality, but it is not a decisive factor in sea-water concrete with which resistance is generally attained by the weight of each element. Likewise, for reinforced concrete, an increase of volume is not of great importance, if costs are well deduced.

Probably the inert silica will be converted into an active constituent, if a high degree of pulverization is attained. However, if fineness of grain favors the chemical combinations of hardening, it likewise will favor also the attack of salt water. If cement is unalterable by itself, a fine grinding is desirable.

Chemical composition is of little importance. Different cements may give similar chemical analyses. Alumina, silica, and lime can be mixed by different methods and can be free or combined. Chalk, quartz, or clay can be added to a cement, thus changing its chemical analysis as desired, but without changing the result. Everything, even errors, can be explained by chemical formulas. Chemistry avails nothing to affirm, but only to deny.

A cement to which had been added a large quantity of well slaked lime, and which was quickly disintegrated in salt water, showed no evidence of swelling in the Le Châtelier testing machine. In another cement which did not show any sign of alteration in boiling water, lime was found by means of the phenolphthalein indicator and by the sulphuric acid test; subsequently, by intensifying the test in hot water, evidence of the existence of free lime finally appeared. This particular cement was made at a high temperature, and the hard burned lime was not hydrated during the time of the ordinary test.

As for blocks, a long delay between their fabrication and submersion is good practice; but this cannot be done with bags of concrete, which form an important type of construction.

Impermeability.—Impermeability is desirable, but the writer does not agree with the opinion so prevalent regarding its advantages. By increasing the area exposed to sea water, disintegration is accelerated, but failure will take place eventually, because on the surface the attack will continue. Impermeability on the surface of the concrete attained by carbonation, has no great value, because many blocks and bags break during the winter. To attempt to keep dry what is always under water is altogether too difficult a task. Even specimens that decompose in laboratory tests are more impermeable and much better made than the best concrete used in construction. Impermeability is only convenient because it retards disintegration and gives time for carbonation; but this procedure is not certain.

Concrete deficient in cement will resist decomposition if the free lime is enclosed by stable elements, or if it can be washed out by the action of waves, leaving a more resistant skeleton. If, on the contrary, lime surrounds the unalterable particles, disintegration will result, notwithstanding the impermeability, unless carbonation proceeds quickly enough to save it.

A block taken out of water, after many years, was as porous as a sponge, the weak elements having been washed out by the waves. Many tests were made on that block, all of which proved that it was stable.

Cast Cement.—The advantage of the great hardness obtained by the use of cast cement is a matter to be studied in each case. The engineer must compare its high price with the possible reduction of volume. As to permanence, only time will prove it; theoretically, the reasons alleged to explain it are not convincing.

An hydraulic index greater than 1 also can be attained with some mixtures of pozzuolana and lime. To be free from lime is not sufficient, because lime may appear afterward. The fact that the cast cement is an aluminate of lime raises many doubts, because lime aluminates are often disintegrated.

It is possible that permanence depends on the process followed in manufacture. The bauxite aids the casting, and, perhaps, under this condition a compound is formed that is unalterable. The manufacturers, however, state that casting is not necessary, and that clinkerization is sufficient. If this is true, it is difficult for the writer to find the reasons for its unalterability. Thus far, however, the facts seem to prove it, but further developments must be awaited.

Pozzuolanas.—Pozzuolanas have been used in Italy from time immemorial. Their use in Germany dates from the time Michaëlis showed that free lime is produced during the setting process.

Some unalterable cements are only mixtures of pozzuolana and lime. The investigations of the writer with cement, lime, and pozzuolana have always proved that pozzuolana is effective in neutralizing a part of the lime by converting it into a stable material.

The action of pozzuolanas is unquestionable. Pozzuolanas shorten the time of setting; if added in sufficient quantities, they make possible a reduction from 6 hours to 1¾ hours.

The tensile strength increases with the quantity of pozzuolana added to the cement or to the lime until the point is reached when all the lime is combined and saturation is attained. Further additions cause the tensile strength to decrease. The hydraulic value of pozzuolana depends on the quantity of soluble silica it contains.

Any method of making artificial pozzuolana is good if a large quantity of active silica results. The best pozzuolana the writer has tried was of volcanic origin with 31% of active silica. The result is bad, if a pozzuolana deficient in active silica is added to a cement. It is bad, also, if cement does not have any free lime as previously defined.

Conclusion.—In order to be stable in sea water, cement must not have any free lime after hardening. A pozzuolana must be added to attain this. For each pozzuolana, investigation must be made to determine the quantity of soluble silica that it contains. It is necessary also to know the quantity of free lime that is contained in the cement after setting. Blending must be in such proportions as to form the mono-calcium silicate, and the mixing must be thorough. In order to attain this, pozzuolana and cement must be mixed before grinding. This is not necessary if the cement has a large quantity of free lime that could combine with the pozzuolana by means of the humidity of the air.

The writer does not pretend to define the chemical reaction that produces setting. It is possible that tri-calcium silicate is not stable in water, and that mono-calcium silicate is then formed; that the setting depends on the crystallization of this silicate; and that the loss of strength, two or three months after setting, is caused by the disintegration of some aluminates.

The microscope indicates, however, that there are many physical phenomena that are very important. It is possible that chemical action plays only a small part in the process, that some combinations are only mixtures, and that hardening results from a particular manner of grouping alumina, silica, and lime.

The writer has found it to be possible by the use of colors to observe the different elements of cement during the long process of hardening. The various porosities of the elements give different intensities of color. Apparently, this is the method that must be followed.

R. FERET,* ESQ. (by letter).†—The writer has the following observations to make on the subject:

It is not believed that priority for cast cement which is highly aluminous, should be conceded to Mr. Spackman, because the products he has turned out, at any rate at the beginning of his researches (the writer does not know whether he has done further research), could not be used by themselves like the cast cement, but had to be mixed with lime or ordinary cements.

The second conclusion of the authors is not quite correct in so far as they state that the additions of pozzuolanas to cements prevent their decomposition by sea water. As a matter of fact, in this respect they produce an appreciable amelioration, but not total or absolute preservation.

* Boulogne-sur-Mer, France.

† Received by the Secretary, July 11, 1923.

IMPROVED TYPE OF MULTIPLE-ARCH DAMS
Discussion*

By Messrs. H. J. F. Gourley and Alfred D. Flinn.

H. J. F. Gourley,† Esq. (by letter).‡—In his studies of the design of multiple-arch dams, the writer has been struck by the weakness of the buttresses in cases where the dam has been more than 100 ft. high. It seemed to be poor design to introduce horizontal struts, sometimes of considerable length in themselves, in order to provide a quite incalculable addition to the strength and stiffness of high buttresses; and it did not inspire confidence in the structure of which such struts formed a part. The logical and economical form of buttress described by the author meets the case admirably.

With a masonry dam of the ordinary gravity type, a portion of the dam is frequently utilized as a spillway. The multiple-arch type does not lend itself so readily to that purpose and, therefore, requires either a separate structure and channel to deal with flood water, or an apron extending over several bays and specially widened buttresses to give a reasonable slope to the apron. In comparing the cost of the two types in any given location, this point must be borne in mind.

The findings of the Committee of investigation on the deflections and stresses in multiple-arch dams mentioned by the author, will be very valuable but, in the meantime, the general method of design proposed appears to be on a sound, although advisedly conservative, basis. Although no multiple-arch dam has been constructed in Great Britain as yet, one has just been completed in Tasmania as part of the Great Lake hydro-electric plan, of which the following brief particulars may be of interest.§

The dam will be 1 182 ft. long and will create a storage of 1 150 000 acre-ft. to supply turbines of 66 000 h. p. capacity, working under a head of 1 200 ft. It is 40 ft. high, and is intended to store water to a depth of 35 ft. above the sill of the sluice-gates with a freeboard of 5 ft. It consists of 27 arches, each of 40 ft. span, and approximately 100 ft. of abutments and cut-off wall. In two places, it was necessary to locate the foundations 50 ft. below the surface for security. Many precautions were required in connection with these excavations, because the Great Lake formed by the old gravity dam was within a few feet of the site.

The arches are inclined at an angle of 60° with the horizontal. The thickness of the arch is uniform in planes normal to the extrados, but increases uniformly at the rate of 0.019 ft. per ft. of depth measured vertically, the crown being 1 ft. at the top. The extrados is a true circle in an horizontal plane, with a radius of approximately 22 ft. The arches were designed in accordance with

* Discussion on the paper by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., continued from November, 1923, *Proceedings*.

† With Sir Alex. Binnie, Son & Deacon, Westminster, S. W. 1, England.

‡ Received by the Secretary, October 10, 1923.

§ *Water and Water Engineering*, July 20, 1923; see, also, *The Electrical Review*, December 8 and 15, 1922.

the elastic theory, and reinforcement was provided over the full span of the intrados and over a part of the extrados near the springing, in order to provide for temperature variation.

The buttresses are tapered and reinforced. They are supported by two lines of beams, 10 ft. 6 in. apart vertically, the tops of the buttresses being tied in by a pair of floor-beams supporting the footway, which runs the whole length of the dam.

No spillway accommodation is provided as evidently it is anticipated that the 5 ft. freeboard will take care of all floods.

ALFRED D. FLINN,* M. AM. SOC. C. E. (by letter).†—Conservative engineers and investors, having become accustomed to the thick sections of "gravity" masonry dams, still look askance at the much thinner sections of the arch and multiple-arch dams that have been advocated and built in recent years. There always will be much individuality in important dams, because of the characteristics of their sites and the differing demands of the several projects. For some projects, local conditions may have the determining influence regardless of the relative merits of different types theoretically compared. For many projects, however, the selection will be guided by the relative economy of the various kinds of dams. An increase in the present knowledge concerning the security of important types, and concerning the best methods for designing and constructing them, is most desirable. For these reasons, Mr. Noetzli and many other members are to be commended for the valuable papers and discussions on this subject, which they have contributed to the publications of the Society during recent years.

Theoretical discussions, no matter how logical or how correct they may be mathematically, gain credence by physical verification. All mathematical computations for such analytically indeterminate structures as dams, begin necessarily with assumptions departing more or less widely (and to unknown degrees), from physical conditions. Quite timely, therefore, are the field and laboratory measurements and tests that are being conducted by the Committee on Arch Dam Investigation, of Engineering Foundation, of which Charles Derleth, Jr., M. Am. Soc. C. E., is Chairman, and Mr. Noetzli is Secretary. A progress report of the work of this Committee has been published,‡ which gives the names of its members and of some of its co-workers. Engineers who have on file temperature records, measurements of deflections and other movements, or any observations on arch or multiple-arch dams, either in service or in course of construction, would aid the Committee in its public-spirited work by submitting this information to its Secretary or Chairman.

A critical study of physical data should give more valid assumptions on which mathematical analyses and formulas for design must be based. The more abundant the reliable data that are put into the hands of the Committee, the better the deductions that it can make, and the more useful to the Profession the final report of the investigation.

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* Director, Engineering Foundation, New York, N. Y.

† Received by the Secretary, November 2, 1923.

‡ See "Items of Interest," p. 644.

STRESSES IN MULTIPLE-ARCH DAMS

Discussion*

By Fred A. Noetzli, Assoc. M. Am. Soc. C. E.

Fred A. Noetzli,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author, deserves much credit for having undertaken the tedious work of investigating the stresses on inclined sections through the buttresses of multiple-arch dams. Some of the calculated stresses differ more or less from those obtained by the usual methods of determining stresses in horizontal planes. Nevertheless, it is gratifying to note that, after calculating the maximum tension, compression, and shear, the author comes to the conclusion that, in the future, higher dams can be built and higher stresses can be used with safety, provided tension is either avoided or taken care of by steel reinforcement.

The structure investigated by the author (see Fig. 33§) has been in successful operation for a number of years, and has stood up well under extraordinarily severe load and temperature conditions, being located in the snow region of the high Sierras and having been overtopped accidentally by floods, without suffering any apparent damage.

For the determination of the foundation pressure, the author recognizes the necessity of calculating the stresses in horizontal sections, inasmuch as normal sections cannot be used for points at the heel of the dam. At the toe, therefore, two different sets of principal stresses are obtained for the same point, *B* (Fig. 33), one if it is considered in a horizontal section, and another if in a normal section. However, as the computed stresses at the down-stream face of a buttress are doubtless in excess of the true stresses, no apprehension should be felt regarding this discrepancy.

It is more important to guard against excessive tension stresses. This can be accomplished in three ways: First, by using conservative values for the permissible stresses in horizontal or normal planes; second, by making the up-stream slope steeper, say, 60 to 65°, so that the normal planes become horizontal, or nearly so; and, third, by adequately reinforcing the buttress walls and arches. In the design of the Horseshoe Dam,‖ the writer has made provision against inclined tensile stresses by reinforcing the arches and buttress walls in vertical and inclined directions. Thus, for the lower parts of the buttresses, a two-way reinforcement is provided consisting of ¾-in. round rods, 18 in. on centers, vertically and diagonally, and, further, consisting of 1½-in. round rods, near the arch groins, extending from the base to the crest of each buttress, thus giving good protection against excessive tension stresses.

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* Discussion on paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from November, 1923, *Proceedings*.

† San Francisco, Calif.

‡ Received by the Secretary, October 17, 1923.

§ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1116.

‖ *Loc. cit.*, p. 1083, Plate III.

The occurrence of cracks in the buttresses of the Lake Hodges Dam, referred to by the author, is easily explained by the fact that the buttress walls of this dam are built of plain concrete, and have no steel reinforcement. It was to be expected, therefore, that temperature and shrinkage cracks would develop in such unreinforced buttresses, the bases of which are in some cases more than 150 ft. long, with no provision for expansion and contraction. The difference in the temperature of the arch barrel and of the buttress walls during certain seasons of the year is considerable, and this may have been one of the main reasons why some of the buttresses have developed cracks. As far as the writer can learn, the width of these cracks is practically independent of the water level in the reservoir and of the load to which the buttresses are subjected. This would indicate that the cracks are not due to the water pressure, but have probably resulted from other causes, such as temperature, and shrinkage.

The position of the crack shown in Fig. 42* is a good illustration of the manner in which the water pressure is transmitted by the buttresses to the foundation. The crack divides the buttress into two parts which may be considered as inclined columns supporting the lower and upper portions, respectively, of the arch barrel. Inasmuch as the cracks have been observed to be slightly open even with the water standing deep in the reservoir, the two inclined columns act for the greater part of their heights practically independently of each other and are only connected near the base by the foundation and at the top by the arch barrel. Evidently, no "horizontal shear" can be transmitted across the open cracks, and the horizontal component of the water pressure is thus seen to be carried to the foundation by the inclined buttress columns. The buttress would appear to be equally strong with a crack and without it, as the crack may have relieved internal shrinkage and temperature stresses, and thus have provided a natural expansion joint. It is likely that temporary or permanent expansion joints, dividing each buttress into two or three inclined columns, would be of advantage from the point of view of eliminating largely the temperature and shrinkage stresses in the buttresses, and would not diminish the stability of such dams. In fact, this idea has been carried one step farther by some engineers in designing dams in which the deck (arches or slabs) are proposed to be supported by inclined struts of suitable dimensions, instead of by buttress walls.

The fact that the buttresses of multiple-arch dams may be considered roughly as consisting of a series of inclined columns, offers an excellent opportunity for constructing such dams in steps by adding a "column" unit at the down-stream side of each buttress when such a dam is to be raised. The lower arches and corresponding parts of the buttresses, of course, must be dimensioned from the start for the ultimate water pressure. Estimates made by the writer, however, seem to indicate that, in many cases, it will be economical to build multiple-arch dams in steps. The double-wall buttress type is particularly suitable for such constructions, inasmuch as the later additions may consist of buttress units of H-column shape, which will act as true columns.

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1128.

It is to be hoped that in the future engineers engaged in dam design will consider the multiple-arch type with the favor which it rightly deserves on account of its great economy and large factor of safety. The severe mathematical test to which the author has subjected the stresses in the buttresses should dispel the last doubts regarding the safety of well designed and well constructed dams of this type.

Regarding Mr. Jakobsen's proposal of making tests to destruction on some fairly high multiple-arch dams, the writer wishes to advise that such tests are being contemplated by the Committee on Arch Dam Investigation which was recently announced by Engineering Foundation. At present, existing arch and multiple-arch dams are being investigated by deflection, stress, and temperature measurements. Full-sized tests will be made as soon as the necessary funds are available, and it is believed that such tests will result in great economies for future dam construction. The Committee would welcome contributions from progressive public utilities and other parties interested in more economical types of dams.

TENTATIVE SPECIFICATIONS FOR STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

SUBMITTED AS A PROGRESS REPORT OF THE SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

Discussion*

BY MESSRS. S. H. LEISTER, THEODORE BELZNER, L. H. SHOEMAKER, AND ADOLF EGGENSCHWYLER.

S. H. LEISTER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The Tentative Specification for Steel Highway Bridge Superstructure, presented by the Special Committee on Specifications for Bridge Design and Construction,§ classifies bridges on the basis of traffic into four groups, as follows:

"Class A.—City bridges or other bridges carrying a highway traffic of exceptionally heavy load units.
"Class B.—Bridges on primary roads.
"Class C.—Bridges on secondary roads.
"Class D.—Bridges carrying electric railway traffic only, or a combination of electric railway and highway traffic."

The only distinction made between the four classes, however, in the remainder of the specification, is in the selection of live loads.

It seems unreasonable to expect country bridges on secondary roads to be designed on the same basis as city bridges with respect to thickness of material, riveted connections, bracing, etc.

Article 301.—If $\frac{1}{4}$ in. is adopted as a minimum thickness of metal for all classes of highway bridges, no exception being made for webs of channels, it would eliminate all the light weight standard channels less than 15 in. in depth. Even a 12-in., 20.7-lb. channel could not be used. The writer believes that an exception should be made from the $\frac{1}{4}$-in. ruling for webs of channels on Class C bridges.

Article 312.—This Article which specifies that no connection, except for lattice-bars and hand-rails, shall contain less than three rivets, and Article 321, which specifies that lateral, longitudinal, and transverse bracing shall be composed of rigid members, are both too severe for Class C bridges, and the writer doubts whether they will be adopted for that class.

Article 310.—If this Article is adopted, it will mean that $\frac{1}{2}$-in. and $\frac{5}{8}$-in. rivets are the largest that will be permissible in 2-in. and $2\frac{1}{2}$-in. angles,

* Continued from September, 1923, *Proceedings*.
† Structural Engr., Tiffin, Ohio.
‡ Received by the Secretary, September 28, 1923.
§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

respectively, if the angles carry calculated stress; $\frac{5}{8}$-in. rivets in 2-in. angles, and $\frac{3}{4}$-in. rivets in $2\frac{1}{2}$-in. angles have been used for many years with satisfactory results, and a change seems unnecessary.

Article 201.—Compression on columns: the gross section is given as "$\dfrac{16.0}{1 + \dfrac{l^2}{13\,500\,r^2}}$, but not to exceed the value for $\dfrac{l}{r} = 40$." Why not state, "but not to exceed 14.0"?

Article 107.—This seems to be a step in the wrong direction. Because of the fact that crowds of people will sometimes congregate on bridge sidewalks and that disastrous failures have occurred as a consequence thereof, the writer thinks that the minimum load for sidewalks should be 100 lb. per sq. ft., instead of 80 lb. per sq. ft.

The general arrangement and scope of the specification are excellent; it will undoubtedly be extensively used.

THEODORE BELZNER,* AFFILIATE, AM. SOC. C. E. (by letter).†—After reading the Tentative Specifications for Steel Highway Bridge Superstructure‡ with much interest the following suggestions are offered:

Article 406.—Rivet Holes.—The writer would suggest that, after reaming or drilling, a special tool should be used over both ends of all rivet holes, to remove burrs and sharp edges, and so shape the holes that a slight fillet would be formed under both heads of all rivets.

Article 428.—Shop Painting.—The writer believes that steel should receive at least two coats of red lead linseed oil paint, which should be mixed on the job and used within 24 hours after mixing, and applied as follows: The first coat shall be darkened with lamp-black and after it has dried not less than 24 hours, and before shipment, a second coat of red lead of bright color shall be applied.

The writer recently had charge of the fabrication of structural material for the reconstruction of a very important bridge and insisted on the red lead paint being used within 24 hours after mixing. Furthermore, no reference is made in the specifications (under "Workmanship") for assembling.

L. H. SHOEMAKER,§ M. AM. SOC. C. E. (by letter).‖—Several highway bridge loadings designed to meet the conditions of modern traffic have recently been proposed. The writer refers to those loadings proposed in the Tentative Specifications for Steel Highway Bridge Superstructure of the Society,‡ in the General Specifications for Steel Highway Bridges of the American Railway Engineering Association, and in the paper by Harold D. Hussey, Esq., entitled "Proposed Loading for Highway Bridges",¶ which will hereafter be referred to as A. S. C. E., A. R. E. A., and Class T-20 Truck.

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* Insp. of Steel, in Chg., Shop Work, Rolling Segments, Reconstruction, Vernon Ave. Bridge, Dept. of Plant and Structures, City of New York, New York, N. Y.

† Received by the Secretary, October 5, 1923.

‡ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

§ Asst. to Chf. Engr., Am. Bridge Co., New York, N. Y.

‖ Received by the Secretary, October 11, 1923.

¶ *Loc. cit.*, August, 1923, p. 1031.

In this discussion, the writer will suggest a loading composed of a line of trucks to be used as a basis for determining a simplified loading consisting of a uniform load per linear foot, together with a single concentrated load at any point, which will have practically the same stress effect.

The considerations which should govern in assuming a loading for a highway bridge are the maximum probable loads which it will be called on to sustain, and the minimum distance apart at which they can move freely at normal speed. As adequate provision is to be made for impact, it is not necessary to consider traffic so congested that it cannot move freely.

The 7½-ton truck, which weighs about 15 tons when loaded to capacity and which can be overloaded to weigh about 20 tons, is the heaviest commonly used. A large part of the traffic, however, is carried in trucks of 5 tons capacity or less. It would not appear to be necessary, therefore, to provide for a succession of 20-ton loads.

In order to provide amply for the condition just described, assume a loading consisting of one 20-ton truck, placed at any point on the structure and followed by, or preceded by, or both, 15-ton trucks spaced 30 ft. apart between axles of adjacent trucks or with a clearance of about 19 ft. This loading hereafter designated as Loading B is shown in Figs. 6 and 7.

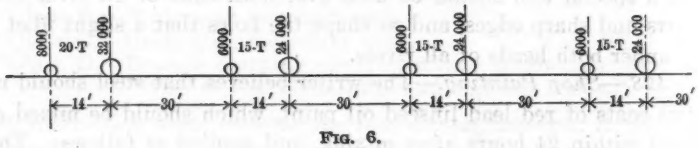

FIG. 6.

It is proposed to devise a simplified loading which will consist of a continuous uniform load covering the loaded length, and a single concentrated load to be placed where it will produce the greatest effect and which can be used in place of Loading B with substantial accuracy as regards the stresses obtained.

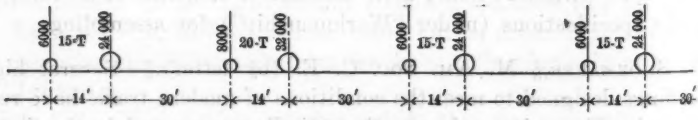

FIG. 7.

Let Figs. 8 and 9 be the influence line diagrams for moments and shear, respectively, for any statically determinate structure.

Let l = the loaded length;
y = the vertical ordinate;
w = the uniform load per linear foot of loaded length;
W = the concentrated load to be placed at C;
S = the stress, either bending moment or shear; and
u = the equivalent uniform load per linear foot.

Therefore,

$$S = \frac{1}{2} w l y + W y$$

$$u = \frac{2S}{ly} = w + \frac{2W}{l}$$

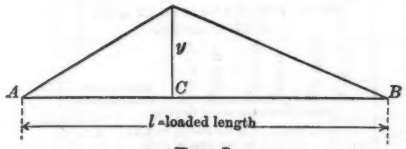

FIG. 8.

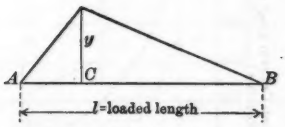

FIG. 9.

It appears, therefore, that the equivalent uniform load is equal to the uniform load plus twice the concentrated load divided by the loaded length, and that it is the same for the same loaded length without regard to the character of stress, whether moment or shear.

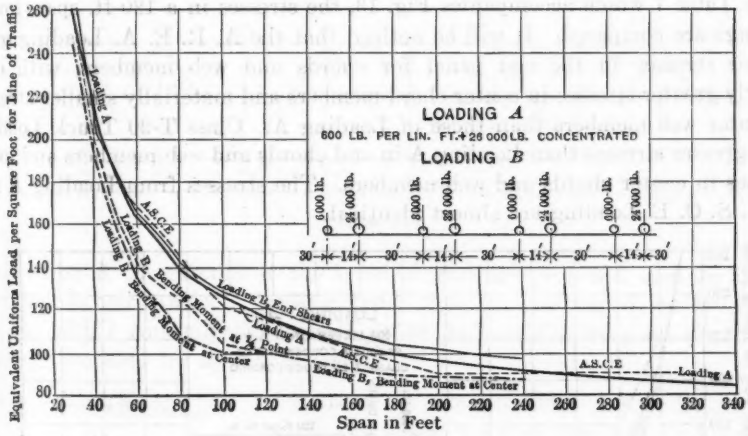

FIG. 10.

A loading of 600 lb. per lin. ft. and a concentrated load of 26 000 lb., to be designated as Loading A, is proposed. In Figs. 10, 11, and 12, this loading is compared with Loading B and the other loadings under discussion on the basis of equivalent uniform loads per square foot for one line of traffic 9 ft. wide.

From Fig. 10, it will be noted that the equivalent uniform loads from Loading A correspond closely to those for Loading B and the A. S. C. E. Loading for loaded lengths of 30 to 250 ft. For loaded lengths of more than 250 ft., Loading A gives results somewhat less than those of the other loadings. From Fig.11, it may be noted that Loading A gives shears materially less than the A. R. E. A. Loading, except for loaded lengths of less than 60 ft. For spans up to 90 ft., Loading A gives greater bending moments at the center than the A. R. E. A. Loading and lower center bending moments for spans of more than 90 ft., with a maximum difference of about 10 per cent. If the equivalent load per square foot had been calculated for a line of traffic 10 ft.

wide according to the A. R. E. A. Specifications, the agreement would have been very close. From Fig. 12, it will be noted that Loading A gives results practically the same as the Class T-20 Loading for bending moments at the quarter-point, larger bending moments at the center, and smaller end shears.

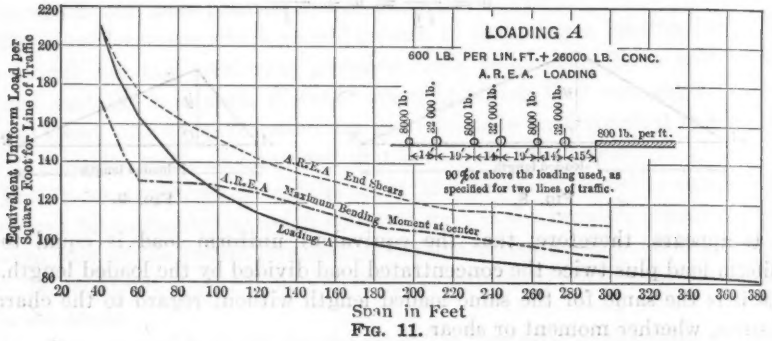
LOADING A
600 LB. PER LIN. FT. + 26000 LB. CONC.
A. R. E. A. LOADING
8000 lb.
22 000 lb.
800 lb. per ft.
90 % of above the loading used, as specified for two lines of traffic.
A. R. E. A. End Shears
A. R. E. A. Maximum Bending Moment at center
Loading A
Equivalent Uniform Load per Square Foot for Line of Traffic
Span in Feet

FIG. 11.

In Table 7 which accompanies Fig. 13, the stresses in a 120-ft. span for all loadings are compared. It will be noticed that the A. R. E. A. Loading gives greater stresses in the end panel for chords and web members, with only slightly greater stresses in center chord members and materially smaller stresses in center web members than those of Loading A. Class T-20 Truck Loading gives greater stresses than Loading A in end chords and web members and lower stresses in center chords and web members. The stresses from Loading A and the A. S. C. E. Loading are almost identical.

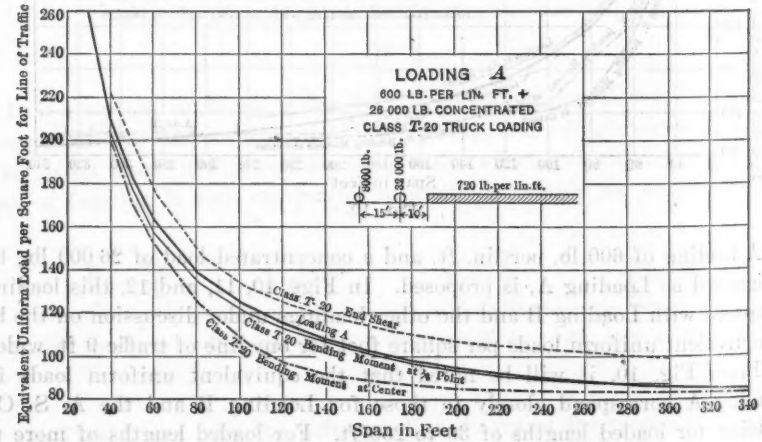
LOADING A
600 LB. PER LIN. FT. +
26 000 LB. CONCENTRATED
CLASS T-20 TRUCK LOADING
8000 lb.
32 000 lb.
720 lb. per lin. ft.
Class T-20 End Shear
Loading A
Class T-20 Bending Moment at ⅜ Point
Class T-20 Bending Moment at Center
Equivalent Uniform Load per Square Foot for Line of Traffic
Span in Feet

FIG. 12.

Loading A as shown gives results as to stress effects that do not vary greatly from those of any of the other loadings and it represents, in a general way, the condition of highway traffic better than the other loadings. It has the advantage of simplicity and ease of application and can be used for loaded lengths as short as 30 ft., as shown by Fig. 10. For shorter spans or loaded lengths, the actual truck load would be used.

TABLE 7.—STRESSES IN 120-FT. TRUSS FOR DIFFERENT HIGHWAY LOADINGS.

(All Stresses in Kips.)

Member.	Loading A.	Loading B.	Percentage of Loading A.	A. S. C. E. Loading.	Percentage of Loading A.	A. R. E. A.	Percentage of Loading A.	T-20 truck.	Percentage of Loading A.
$L_0\ L_2$	+40.7	+37.0	91	+40.4	100	+46.7	114	+41.4	101
$L_2\ L_4$	+87.2	+73.6	84	+86.6	99	+88.6	102	+81.0	93
$U_1\ U_2$	−69.8	−60.4	87	−69.2	99	−80.6	115	−69.3	100
$U_3\ U_4$	−93.0	−81.2	87	−92.3	99	−101.3	109	−79.5	85
$U_1\ L_0$	−67.8	−65.2	96	−67.3	99	−78.3	115	−69.0	102
$U_1\ L_2$	+53.3	+50.3	94	+51.7	97	+59.4	111	+52.8	99
$U_3\ L_2$	−40.4	−38.9	96	−41.3	102	−41.9	104	−38.3	95
"	+11.3	+11.3	100	+11.3	100	+10.2	90	+11.3	100
$U_3\ L_4$	+29.2	+30.5	104	+30.4	104	+27.0	92	+25.6	87
"	−19.4	−17.6	91	−19.4	100	−15.8	82	−17.6	91

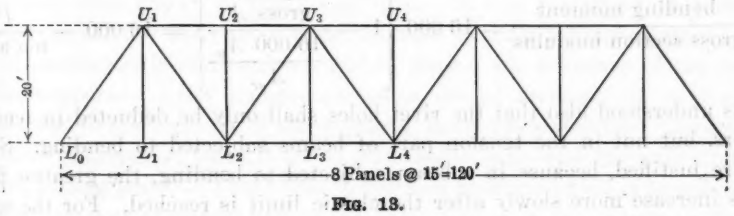

FIG. 13.

Loading A corresponds to the heaviest loadings proposed, and for bridges carrying lighter traffic, a loading using 75% of Loading A, or 450 lb. per lin. ft., with a concentrated load of 19 500 lb., would correspond closely with loadings proposed for secondary traffic.

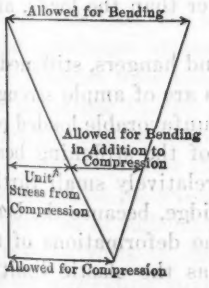
Allowed for Bending
Allowed for Bending in Addition to Compression
Unit Stress from Compression
Allowed for Compression

FIG. 14.

ADOLF EGGENSCHWYLER,* ASSOC. M. AM. SOC. C. E. (by letter).†—With reference to Article 121, "Combined Stresses",‡ it is stated that "Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the unit stresses given in Article 201." If compression and bending are combined, Article 201‡ gives two different unit stresses, one for compression and one for bending. Which shall be used? The writer would suggest a straight-line interpolation as shown by Fig. 14 and would propose the following: The bending stress in addition to compression shall not exceed the allowed bending stress without compression, multiplied by $\left(1-\frac{\text{real compression}}{\text{allowed compression}}\right)$, or according to the Tentative Specifications:

* St. Paul, Minn.

† Received by the Secretary, October 18, 1923.

‡ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1384.

$$f_b = \frac{\text{bending moment}}{\text{gross section modulus}} \leq 16\,000 \left\{ 1 - \frac{\dfrac{P}{\text{gross area}}}{\dfrac{16\,000}{1 + \dfrac{l^2}{13\,500\,r^2}}} \right\}$$

$$= 16\,000 - \frac{P\left(1 + \dfrac{l^2}{13\,500\,r^2}\right)}{\text{gross area}}.$$

If tension and bending are combined, the Tentative Specifications allow 16 000 lb. per sq. in. tension in the net section, or $16\,000 \times \frac{\text{net area}}{\text{gross area}}$ in the gross section, and, for bending, 16 000 lb. per sq. in. in the gross section. The writer likewise would propose in a manner analogous to that for compression and bending, the following equation:

$$f_b = \frac{\text{bending moment}}{\text{gross section modulus}} = 16\,000 \left\{ 1 - \frac{\dfrac{P}{\text{gross } A}}{\dfrac{16\,000\,A_n}{A_{gr}}} \right\} = 16\,000 - \frac{P}{\text{net area}}.$$

It is understood also that the rivet holes shall only be deducted in tension members, but not in the tension part of beams subjected to bending. Such rules are justified, because in a beam subjected to bending, the greatest fiber stresses increase more slowly after the elastic limit is reached. For the same fiber stress, a beam has, therefore, a greater safety factor than a tension member.

With reference to Article 201, the writer would suggest that different unit stresses be allowed, depending on whether or not a relatively small excess of loads that produce stresses equal to the elastic limit would have severe consequences, that is, whether the unit stresses increase faster or slower than the loads, after the elastic limit is exceeded.

Fig. 15 shows an arch with pin-connected members and hangers, stiffened by a straight beam. Assume that the arch and the hangers are of ample strength, and that a certain live load, p, distributed over the most unfavorable loaded portion, a, produces a fiber stress in the cross-section, x, of the stiffening beam, equal to the elastic limit. Under these conditions, a relatively small further increase of the load, p, will cause the collapse of the bridge, because the bending moment in the cross-section, x, is increased by the deformations of the truss, this influence increasing very rapidly as soon as the elastic limit is exceeded in a part of the stiffening beam.

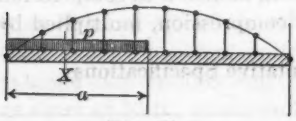

FIG. 15.

A correspondingly stiffened suspension bridge, Fig. 16, however, would never collapse due to the weakness of the stiffening beam, even when a much smaller load than the theoretical maximum, if distributed over the most unfavorable loaded portion, a, would stress the stiffening beam in the cross-section, x, to the elastic limit. In this case, the deformations of the truss

decrease the bending moments in the beam, and after the elastic limit is reached in the beam, the fiber stresses will increase much more slowly than the loads.

In Fig. 15, the stiffening beam is necessary in order to prevent the collapse of the bridge due to partial loading, and it would be very dangerous if the bending stresses in the stiffening beam approached or exceeded the elastic limit, whereas, in Fig. 16, the stiffening beam is merely used for decreasing the undesirable deformations of the truss, but is not necessary for the stability of the structure. An occasional exceeding of the elastic limit, due to extraordinary partial loadings, would have no serious consequences. It would be justifiable, therefore, to allow for the stiffening beam in Fig. 16 much higher fiber stresses than for that in Fig. 15.

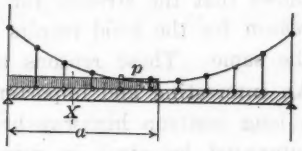

FIG. 16.

There is also an obvious discrepancy between the low allowed unit stresses in structural steel bridges and the high working stresses in railway rails. The high working stresses in railway rails have been shown by recently published tests* and were known to many engineers. It was customary to allow fiber stresses for railway rails about equal to those used for structural steel bridges, but to consider only statical loads and to neglect the influence of the yielding of the supports. After mentioning some liberal assumptions in regard to impact and to the yielding of the supports, Röll† has stated that "By considering these unfavorable facts, it seems permissible to go with the unit stresses up to the yielding point". This refers to German practice which is to reduce carefully the sources of impact and to use heavier rails, in proportion to the wheel loads, than is customary in the United States, which results in considerably less breakage of rails.

If a railway rail was designed by considering the greatest vertical and lateral impacts, together with the influence of the yielding of the ties over hard or soft ground and of the fact that some ties may be badly bedded or partly decayed and that the spaces between them become irregular by service, and if the maximum stresses due to the most unfavorable combination of these circumstances were calculated and 50% of the highest stresses with opposite sign occurring in the same place was added, the stresses would always far exceed the elastic limit. In other words, if the stresses allowed for railway rails were no greater than those for structural steel bridges, the weight of the rails would be increased about ten times. Yet, in practice, railway rails have proved to be of sufficient strength. The good results obtained with high stresses in railway rails is due to the fact that, in such continuous beams with elastically yielding supports, the fiber stresses increase much more slowly than the loads after the elastic limit has been exceeded. Not only does the maximum fiber stress increase more slowly than the bending moment, but, due to impact and to the influence of the yielding of the supports, the bending moments also increase more slowly than the loads, if the elastic limit is exceeded.

* Progress Reports of the Special Committee on Stresses in Railroad Track published by the Society.

† Encyclopädie des Eisenbahnwesens", Bd. 7, S. 417.

Experience and tests have also shown that other devices, such as chain links and shackles, are safe for certain working loads, whereas a strict calculation shows that the stresses for these loads must exceed the elastic limit. The reason for the good results with such devices and railway rails is generally the same. These reasons also hold good for many parts of steel bridges. An interesting and important example is the steel pontoon bridge. Assume a long pontoon highway bridge with long, continuous, steel, main girders, supported by steel or reinforced concrete pontoons spaced about 80 ft., interrupted perhaps by a movable part for navigation, and connected to the shores by single-beam bridges. Reasonable cross-sections will be obtained if the main girders are designed by assuming rigid supports. If, however, the yielding of the pontoons is considered, and the greatest possible bending moments of the different cross-sections due to partial loading are increased by 50% of the greatest possible bending moment with opposite sign occurring in the same cross-section, the stresses are much higher. By increasing the strength of the beams, the waves of the influence lines for the bending moments become longer. The greatest bending moments due to partial loading are much increased and the fiber stresses decrease, therefore, in a much smaller ratio than the weight of the beams is increased. Absurdly heavy cross-sections would be required in order to reduce the fiber stresses to the figure generally allowed for steel bridges.

Such pontoon bridges would probably prove to be sufficiently strong, if the yielding of the supports was not considered, or if, by considering it, stresses up to the elastic limit were permitted, and more liberal assumptions allowed in regard to partial loading by a crowd than for beams according to Fig. 15.

In the writer's opinion it would be justifiable, therefore, to differentiate between the unit stresses allowed for steel bridges according to whether or not an excess of the elastic limit in the affected part is dangerous for the stability of the structure, that is, whether the stresses, after exceeding the elastic limit, increase more quickly or more slowly than the loads. The generally allowed unit stresses are justified for cases of the first type, for instance, for stiffening beams according to Fig. 15, whereas, stiffening beams according to Fig. 16 and continuous pontoon bridge girders should preferably be brought in the same class with railway rails, chains, and shackles for which experience and tests have shown that much higher fiber stresses are safe.

With reference to Article 204,* "Compression Members", the writer would suggest the use of the Strand-Euler formula, Euler for $\frac{l}{r} > 140$ and Strand for $\frac{l}{r} < 140$. These are the only formulas that can be called a scientific solution of the problem, and they agree much better with tests than the other compression formulas. Other compression formulas are merely attempts to represent the graphically plotted results of tests by a curve corresponding to as simple a mathematical formula as possible and are based on the Euler formula inasmuch as they assume that the ultimate compression strength is a function of

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1385.

$\frac{l}{r}$. As far as they refer to members with $\frac{l}{r} > 100$, they have never been necessary, because they are always less accurate and never simpler than the Euler formula. For members with $\frac{l}{r} < 100$, they were useful as long as a proper mathematical solution was not available, but there is no need of their further use since Strand succeeded in finding the solution.* Diagrams annexed to Strand's publication show an astonishing agreement between his mathematical derivations and the average test results of Tetmajer, for various materials, such as structural steel, cast iron, and timber.

The objection could be made that the Strand formula requires the use of logarithms and cannot be solved with the slide-rule alone. This, however, is not a vital defect, because the Strand formula can easily be shown by means of a diagram from which the ultimate strength can be obtained at a glance for any ratio of $\frac{l}{r}$.

* *Zentralblatt der Bauverwaltung*, 1914, p. 88*ff*, and "Die Knickfestigkeit", by Rudolf Mayer, Berlin, 1921.

REACTIONS FOR A PARTICULAR TYPE OF UNSYMMETRICAL ARCH

Discussion*

By Messrs. Victor H. Cochrane and J. Charles Rathbun.

Victor H. Cochrane,† M. Am. Soc. C. E. (by letter).‡—Any procedure that will decrease the labor of designing arches, is welcome. The relationship shown by the author, between a particular kind of unsymmetrical arch and the symmetrical arch from which it is derived, may be the means of greatly lessening the work of designing an unsymmetrical arch, as will be shown.

In a paper published in 1916,§ the writer developed short-cut methods for the design of symmetrical arches. They were intended for use in preliminary calculations, but have been found to give sufficiently accurate results for final designs in many cases, and are widely used for that purpose. Among the many letters received by the writer, relating to these methods, there were two or three in which it was asked whether they can be applied to the design of unsymmetrical arches. One correspondent stated that he had designed arch spans on the assumption that the two parts of the unsymmetrical arch, separated by an imaginary vertical section at the highest point of the axis, could be treated as the halves of symmetrical arches having equal horizontal thrusts due to the dead load. On this basis, the arch axis will be of nearly the same shape as that obtained by the author's method of parallel projection, provided the crown is located by the formula:

$$d = \frac{l\,h}{8\,r} \qquad (18)$$

in which

d = horizontal distance from center line of span to crown;
l = span length;
h = difference in elevation of ends of arch; and
r = vertical rise of arch at center line of span.

This method will give good results when h and d are small, as compared with the span length; but in an extreme case, such as that of a half-arch with the crown-point at the springing line $\left(d = \frac{l}{2}\right)$, the arch could not be designed

* This discussion (of the paper by Carl B. Andrews, Assoc. M. Am. Soc. C. E., published in October, 1923, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Tulsa, Okla.

‡ Received by the Secretary, October 15, 1923.

§ *Proceedings*, The Engineers' Soc. of Western Pennsylvania, Vol. 32 (November, 1916). The greater part of this paper will be found in "Concrete Engineers' Handbook", by Hool and Johnson, pp. 669-691, and there is a full discussion of the subject in Warren's "Engineering Construction", Part II.

correctly as the half of a symmetrical arch, because the thickness at the crown end would be too small as compared with that at the lower end. The thicknesses required at the two ends will usually be about the same.

For the type of unsymmetrical arch assumed by the author, he has demonstrated that, neglecting the effect of axial stress, the moments at the supports and the horizontal thrust will be the same as for the symmetrical arch; also, that the vertical reactions for the two arches will differ by an amount equal to the product of the horizontal thrust and the tangent of the angle of inclination of the chord of the unsymmetrical arch, or $H \times \frac{h}{l}$. These facts can be shown graphically in a very simple manner, as follows:

In Fig. 4 (*a*), the line, *A B C*, represents a symmetrical, open-spandrel arch, and the line, *A B' C'*, the corresponding unsymmetrical arch. The dimensions chosen are such that the rise is equal to three-tenths of the span, and the distance, *C C'*, is one-half the rise. The axis, *A B C*, is laid out according to the formula:

$$y = \frac{8\,r\,l}{6 + 5\,r}\,(3\,c^2 + 10\,c^4\,r)^* \qquad (19)$$

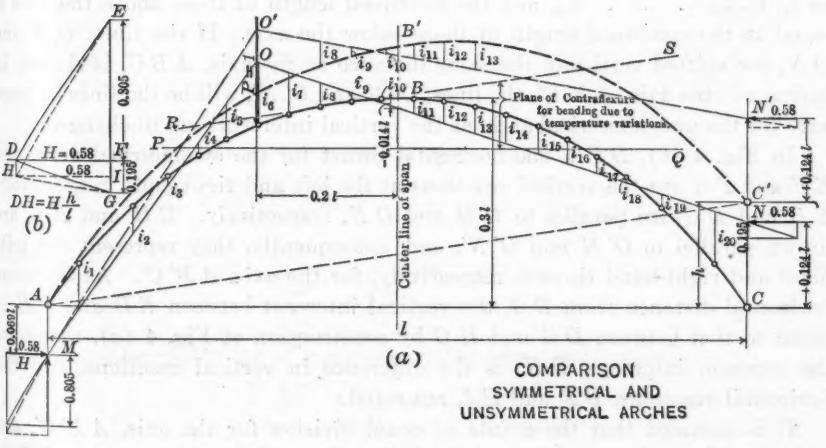

FIG. 4.

The corresponding formula for filled-spandrel arches is:

$$y = \frac{4\,r\,l}{1 + 3\,r}(c^2 + 24\,c^5\,r) \qquad (20)$$

For the arch of Fig. 4 (*a*), the formula reduces to:

$$y = 0.96\,l\,(c^2 + c^4).$$

The arch is assumed to vary in thickness according to Type $A_{2\cdot0}$,† so that the thickness at the springing line is twice that at the crown. Assuming a

* "Concrete Engineers' Handbook", p. 669.

† *Loc. cit.*, p. 671.

unit load, P, at some point on the symmetrical arch, as at a distance of $0.2\,l$ to the left of the center, the lines of thrust can be determined from Figs. 3 and 6.* From these diagrams,

Horizontal thrust (H_c)	0.58
Reaction at right support (V_c)	0.195
Reaction at left support	0.805
Moment at crown (M_c)	$-0.008\,l$
Moment at left springing	$-0.040\,l$
Moment at right springing	$+0.072\,l$
Normal thrust at left springing	0.99
Normal thrust at right springing	0.48

The lines of thrust due to the unit load are shown on Fig. 4 (a) as $O\,M$ and $O\,N$. If the arch axis is divided into twenty sections, in such a manner that the ratio of length of section to the moment of inertia of the arch at the center of the section is a constant, the location of the centers of these sections will be as indicated along the axis. It is well known that the algebraic sum of the vertical intercepts between the axis and the lines of thrust, at the division points, will then be equal to zero. These intercepts are shown on Fig. 4 (a) as i_1, i_2, i_3, . . . i_{20}, and the combined length of those above the axis is equal to the combined length of those below the axis. If the lines, $O\,M$ and $O\,N$, are shifted vertically the same distance as the axis, $A\,B\,C$, is shifted in laying out the axis, $A\,B'\,C'$, the lines, $O'\,M$ and $O'\,N'$, will be the lines of pressure for the unsymmetrical arch, as the vertical intercepts are unchanged.

In Fig. 4 (b), $D\,F$ is the horizontal thrust for the symmetrical arch, and $E\,F$ and $F\,G$ are the vertical reactions at the left and right ends, respectively. $E\,D$ and $D\,G$ are parallel to $O\,M$ and $O\,N$, respectively. $E\,H$ and $H\,G$ are drawn parallel to $O'\,M$ and $O'\,N'$, and, consequently, they represent the left-hand and right-hand thrusts, respectively, for the axis, $A\,B'\,C'$. At the same horizontal distance from $E\,G$, the vertical intercept between $E\,D$ and $E\,H$ is equal to that between $D\,G$ and $H\,G$ by construction of Fig. 4 (a), therefore, the common intercept, $D\,H$, is the difference in vertical reactions, and the horizontal reactions, $D\,F$ and $H\,I$, are equal.

It is assumed that the points of equal division for the axis, $A\,B'\,C'$, are vertically above those for the axis, $A\,B\,C$, and in order to insure this relationship, it is necessary to increase the thickness of the arch, $A\,B'\,C'$, toward the left springing line, at a greater rate than toward the right springing line. This is as it should be, as the thrusts are greater at the left than at the right springing line. However, if the two arches are made of the same thickness, as nearly as may be, the effect on the thrusts and moments will be relatively slight.

The following is the development of a formula for the thickness of an unsymmetrical arch, such that the points of division will be at the same horizontal distance from the center line of span as the corresponding points on the symmetrical arch. In Fig. 5, let,

* "Concrete Engineers' Handbook", pp. 674 and 676.

t_0 = thickness of symmetrical arch at center line of span;

t'_0 = thickness of unsymmetrical arch at center line of span;

δs and $\delta' s$ = segments of symmetrical and unsymmetrical axes, respectively, projected from the same horizontal distance, δx;

t_x and t'_x = thickness of the arches at the centers of the segments, δs and $\delta' s$, respectively;

α = the angle of inclination of the chord of the unsymmetrical arch axis $\left(= \tan^{-1} \frac{h}{l} \right)$;

Δ = the ratio, $\frac{\delta s}{I}$, for the symmetrical arch; and

Δ' = the corresponding ratio for the unsymmetrical arch.

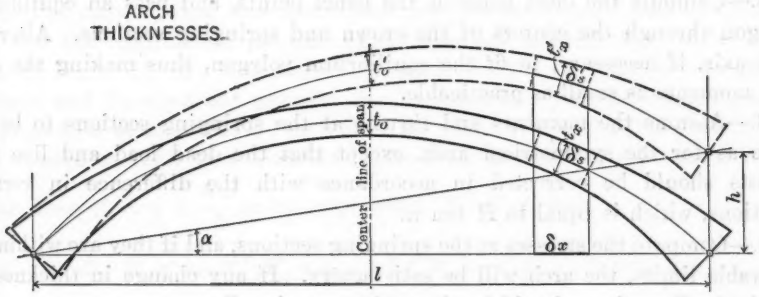

FIG. 5.

At the center line of the span, for an arch of unit width:

$$\Delta = \frac{12\ \delta\ s}{t_0^{\ 3}}, \text{ and } \Delta' = \frac{12\ \delta'\ s}{t'_0{}^3} = \frac{12\ \delta\ s \sec \alpha}{t'_0{}^3};$$

therefore,

$$\frac{\Delta'}{\Delta} = \left(\frac{t_0}{t'_0}\right)^3 \sec \alpha.$$

At any other point the $\Delta' s$ must have the same ratio, therefore,

$$\frac{12\ \delta\ s}{t_x^{\ 3}} \cdot \left(\frac{t_0}{t'_0}\right)^3 \sec \alpha = \frac{12\ \delta'\ s}{t'_x{}^3},$$

whence,

$$t'_x = t_x \sqrt[3]{\frac{\delta' s}{\delta s} \cos \alpha}\ \frac{t'_0}{t_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Ordinarily, t'_0 will be assumed equal to t_0.

If the arch thicknesses are related according to Equation (21), the moment at any point in the unsymmetrical arch, due to a load at any point on the span, is equal to the moment at the corresponding point (vertically below) in the symmetrical arch. Thus, the moments are equal, not only at the supports, as stated by the author, but also throughout the span.

In Fig. 4 (*a*), the line, *P Q*, represents the plane of contraflexure for bending due to temperature variations, or to the shortening of the arch under load,

and $R S$ is the corresponding line for the unsymmetrical arch. If the vertical intercepts between either of these lines and the corresponding arch axis at the division points are considered of opposite sign when measured in opposite directions from the line, their algebraic sum will be equal to zero. Therefore, the temperature thrusts and moments will be the same for both arches, and the arch shortening thrusts and moments may be assumed to be the same.

If an unsymmetrical arch is to be adapted from a previously designed symmetrical arch, as in the case of a bridge with several equal spans, the following procedure may be followed:

1.—Plat the axis of the unsymmetrical arch in the manner described by the author, and lay out the intrados and extrados according to the thicknesses given by Equation (21).

2.—Compute the dead loads at the panel points, and pass an equilibrium polygon through the centers of the crown and springing sections. Alter the arch axis, if necessary, to fit the equilibrium polygon, thus making the dead load moments as small as practicable.

3.—Assume the moments and thrusts at the springing sections to be the same as for the symmetrical arch, except that the dead load and live load thrusts should be corrected in accordance with the difference in vertical reactions, which is equal to $H \tan \alpha$.

4.—Compute the stresses at the springing sections, and if they are within the allowable limits, the arch will be satisfactory. If any change in thickness is required, all sections should be changed proportionally.

If there is no symmetrical arch on which to base the design, the methods previously mentioned may be used to prepare a preliminary design of the symmetrical arch, the subsequent procedure being the same as that already outlined.

J. Charles Rathbun,* M. Am. Soc. C. E. (by letter).†—The value of this paper depends on the amount of reduction in the labor of analyzing the unsymmetrical arch ring through the use of the author's method. It would be of interest to know the percentage of this saving. As arch design requires long arithmetical computations, any research that advances the theory so as to reduce this labor is a boon to engineers.

A proof of the case of shearing distortion, other than the algebraic one given, may be obtained by a study of Fig. 2.‡ Each point of the equilibrium polygon for Arch No. 1 and each point of its corresponding force polygon may be referred to rectangular co-ordinates, one vertical and the other horizontal. If new axes of reference are used and the diagram is redrawn, using the system of co-ordinates shown in Fig. 6, but with values obtained from Arch No. 1, the equilibrium polygon, Arch

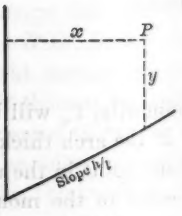

Fig. 6.

* Asst. Prof., Civ. Eng., Univ. of Washington, Seattle, Wash.
† Received by the Secretary, October 26, 1923.
‡ *Proceedings*, Am. Soc. C. E., October, 1923, p. 1688.

No. 1-*A*, and its force polygon are reproduced, provided the moment of inertia of the arch ring is adjusted so that the value of Δ is unchanged and provided, further, that the loads are vertical.

To prove this statement it must be shown that the transformation does not destroy the existing relationship between the force and equilibrium polygons, and that the fundamental relations between the equilibrium polygons and the arch ring, as quoted by the author,* remain true after transformation.

Before its projection, each ray of the force polygon and the corresponding side of the equilibrium polygon are parallel, that is, meet at an infinite distance away. The method of projection does not make this distance finite, and the lines remain parallel. This is the necessary and sufficient relationship between the force and equilibrium polygons.

As the value of y for each point does not change during transformation, the segment of each vertical line intercepted between the neutral axis of the arch ring and the equilibrium polygon (δy), does not change; but, $M_x = H\,\delta y$. In Arch No. 1, $\Sigma_0^{\,l} M_x = 0$, whence, in Arch No. 1-*A*, $\Sigma_0^{\,l} M_x' = 0$ as H is a constant and δy does not change. By hypothesis, x remains unchanged by the projection as does also y; therefore, $\Sigma_0^{\,l} M_x' x = 0$ and $\Sigma_0^{\,l} M_x' y = 0$. If it is desired to refer to a horizontal axis after projection, substitute, as the author does, $y' = y + \frac{h\,x}{l}$ in the expression, $\Sigma_0^{\,l} M_x y$, and,

$$\Sigma_0^{\,l} M_x y' = \Sigma_0^{\,l} M_x y + \frac{h}{l} \Sigma_0^{\,l} M_x = 0 + \frac{h}{l} \times 0 = 0$$

is obtained.

This proves that the equilibrium polygon, No. 1-*A*, is the true one for this arch and that the force polygon, No. 1-*A*, is its force polygon. The values of M_x for all points, including the abutments, are unchanged, and the value of H, a horizontal distance in the force polygon, as well as of the load line, is also unchanged. The vertical position of the pole relative to the load line is lowered a distance equal to $\frac{H\,h}{l}$, where $\frac{h}{l}$ is the tangent of the angle through which the horizontal axis is rotated and this same distance, to scale, corresponds to the amount of change in the vertical reactions.

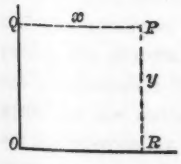

FIG. 7.

Even a more general transformation may be shown to be valid. Assume a symmetrical arch ring, its equilibrium polygon and its force polygon lying in a plane, $z = 0$. Point P of Fig. 7 is any point on these polygons having co-ordinates, x and y, referred to rectangular axes. Draw the Z-axis through the origin in any direction (not of necessity perpendicular to the plane, $z = 0$). Through the points of the polygons, draw projection lines parallel to the Z-axis and pass any plane cutting these lines. The system of points thus obtained on this plane will form a new equilibrium polygon, a new force polygon, and a new arch ring provided, as before, the dimensions at each point of the new ring are chosen so as to give the proper moments of inertia.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1696.

To prove this, consider the rectangle, $O\,Q\,P\,R$ (Fig. 7). This is transformed into a parallelogram, $O'\,Q'\,P'\,R'$ (Fig. 8), in which the angle, $Q'\,O'\,R'$, and the ratios, $\frac{y'}{y}$, $\frac{x''}{x}$, $\frac{x'}{x''}$, $\frac{x'}{x}$, are constant, regardless of the location of P and its corresponding point, P', for any given cutting plane and Z-axis. Let

$$\phi = \text{angle } Q'\,O'\,R',$$

$$a = \frac{y'}{y};\ b = \frac{x''}{x};$$

and,

$$\cos\phi = \frac{x'}{x''}.$$

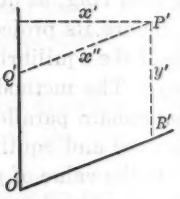

FIG. 8.

It follows that:

$$x' = x'' \cos\phi = b\cos\phi\, x;$$

$$y' = a\,y;$$

$$\delta y' = a\,\delta y;$$

(*a*) $H' = b\cos\phi\, H$, if the load is increased by the proportion, a; or,

(*b*) $H' = \frac{b}{a}\cos\phi\, H$, if the load is unchanged, but the scale of the force polygon is changed;

(*a*) $M_x' = a\,b\cos\phi\, M_x$; or,

(*b*) $M_x' = b\cos\phi\, M_x$;

and,

$$\Sigma M_x' = b\cos\phi\,\Sigma M_x = 0;$$

$$\Sigma M_x'\,x = b^2\cos^2\phi\,\Sigma M_x = 0;$$

$$\Sigma M_x'\,y = a\,b\cos\phi\,\Sigma M\,y = 0.$$

The transformation leaves the fundamental relationship between the equilibrium polygon and the arch ring unchanged; the force polygon is still the one that will produce the given equilibrium polygon. If $b\cos\phi = a = 1$, the case discussed by the author is obtained.

An apparent disadvantage to the method of analysis is that the ring must be made symmetrical by projection. This limits its application materially. In practice, there is little objection if a slight adjustment only in the shape of the arch ring is required to meet this limitation.

PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII

Discussion*

By C. E. Grunsky, M. Am. Soc. C. E.

C. E. Grunsky,† M. Am. Soc. C. E. (by letter).‡—The author is correct when he states "that many have suspected a relationship between solar cycles and terrestrial rainfall". A notable example is Father Ricard of Santa Clara, Calif., who for some time has been making long-range weather forecasts based on the number, size, and location of sun-spots. The writer is not prepared to say with what success this is done and confesses to having taken but little stock in the theory that activities in the atmosphere of the sun can modify in any material degree the quantity of rain which falls on the surface of the earth. The total annual evaporation from the water and land surface of the earth is fairly uniform from year to year and, in consequence, it seems reasonable to assume that the annual precipitation the world over will also be fairly uniform. Furthermore, the air movements are such that, in a general way, the return to the earth of the surplus moisture in the air in the form of rain or snow will occur in a similar manner from year to year, although, of course, with departures in one direction here and there, offset by departures in the opposite direction at other points. For this reason it is hardly to be expected that any general law of increase or decrease of precipitation on the earth can ever be correlated with the solar phenomena. If there is any effect due to the activities in the sun in any measure controlling the movement of storm areas, with or without effect on local precipitation, it still remains to be demonstrated.

It is to be noted, however, that, suspecting such an inter-relation, meteorologists in various parts of the world have set themselves to prove that there is a cycle coincident with the sun-spot cycle and also another equal to one-ninth of the sun-spot period. It is already claimed, in this connection, that the available observations taken from many parts of the world show that this latter period, the general length of which is between 14 and 15 months, is a real one, not attributable to accidental causes. The cycle, three times as long, suggested by the author, and the influence of the sun's activity which he suspects, are not proven by the records which he cites. The data he has used are inadequate to make the demonstration conclusive. His contribution should stand, therefore, for the present, as an interesting example of the method of applying

* This discussion (of the paper by Joel B. Cox, Assoc. M. Am. Soc. C. E., published in October, 1923, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

‡ Received by the Secretary, October 15, 1923.

the law of probabilities to certain problems, rather than as a demonstration of cycles of rainfall intensity.

Great variations in the quantity of rain that falls in a period as short as a calendar month are possible, due to causes that may be considered accidental, as when rainfalls of exceptional intensity occur. Attention may be directed in this connection to Fig. 6* in which the 3.7-year cycle is not nearly as apparent in the 36-year record line as in the lines based on the 16-year period. It would be interesting to know whether the other 20 years considered by themselves also indicate the same cycle. For the 10-year period, 1911 to 1920, in which the mean monthly rainfall was between 15 and 16 in., the author shows an average improvement of ½ in. per month in the monthly forecasts that are based on a recognition of the cycle. This improvement may well be only accidental, or, perhaps, it may be due, in part, to the fact that the rainfall records of this period helped to define the cycle. The fact that in 76 out of the 120 months of the 10-year period a somewhat closer approximation to the actual precipitation could be made under the assumption of the 3.7-year cycle, than would have resulted from forecasts based on the normal monthly rainfall, is not a conclusive demonstration of the existence of such a cycle.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1707.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM COULSON ARMSTRONG, M. Am. Soc. C. E.*

DIED JUNE 12, 1923.

William Coulson Armstrong, the son of John and Elizabeth Armstrong, was born in Marshall County, Iowa, on June 21, 1859. After obtaining an elementary training on the farm, in the rural schools, and as a school teacher, he began his engineering education at Iowa State College, at Ames, Iowa, in 1878. He made a splendid record in college and was graduated with honors in 1881, receiving the Bachelor's Degree in Civil Engineering. In 1904, he obtained the professional degree of Civil Engineer, and, in 1920, was awarded the Honorary Degree of Doctor of Engineering by the College.

Following his graduation, Mr. Armstrong began the practice of his profession, and a record of the positions that he filled is as follows: During 1882 and 1883, Transitman and Draftsman with the Wisconsin, Iowa and Nebraska Railroad Company (now the Chicago Great Western Railroad Company); during 1884, Resident Engineer on the Burlington, Cedar Rapids and Northern Railroad in Dakota; during 1885, with the Northwest Publishing Company, compiling county maps in the State of Minnesota; during 1886, Resident Engineer on the Chicago, Milwaukee and St. Paul Railway in Dakota; in 1887, Resident Engineer on the Minneapolis, St. Paul and Sault Ste. Marie Railway in Minnesota; from 1888 to 1892, inclusive, Resident Engineer on the Great Northern Railway on various lines under construction, and Engineer of Track and Bridges on the Pacific Coast Extension; during 1893 and 1894, with the Andrews Bridge Company, at Spokane, Wash.; from 1895 to 1898, inclusive, Draftsman and Designing Engineer with the Toledo Bridge Company, at Toledo, Ohio; from 1899 to 1901, Resident Engineer, Chicago and North Western Railway Company, at Boone, Iowa, on the construction of a viaduct across the Des Moines River; from 1902 to 1904, inclusive, Bridge Engineer of the Chicago and North Western Railway Company; 1905, Superintendent of Construction, Missouri Pacific Railroad Company; 1906, Bridge Engineer of the Chicago, Rock Island and Pacific Railroad Company; from 1907 to 1912, Terminal Engineer, Chicago and North Western Railway Company, on the design and construction of the passenger terminal in Chicago, Ill.; during 1912 and 1913, Resident Engineer, Chicago and North Western Railway, on the location of a part of an extension from Peoria to Girard, Ill.; during 1913 and 1914, Bridge Engineer, Chicago and North Western Railway; and from 1915 to 1923, inclusive, Chief Engineer of the St. Paul Union Depot Company, on the design and construction of the Union Passenger Station at St. Paul, Minn.

* Memoir prepared by a Committee consisting of Adolph F. Meyer, *Chairman*, E. C. Carter, G. H. Hutchinson, Paul C. Gauger and Bernard Blum, Members, Am. Soc. C. E., and George W. Hand, Assoc. M. Am. Soc. C. E.

Mr. Armstrong's experience was notably extensive and varied. During the earlier years, he obtained an intimate knowledge of the problems of railroad location, construction, and operation, which was valuable and which he applied in his later work. The Boone Viaduct, the North Western Passenger Station, and the St. Paul Union Depot are all engineering works of outstanding magnitude, and each bears evidence of his industry and skill as an engineer.

As Resident Engineer of the Boone Viaduct, he had charge in the field of the erection of this structure. As Terminal Engineer of the Passenger Station in Chicago, he directed the complicated surveys, supervised the drawing of the plans for the masonry work, the bridges, and the train-shed, and worked out the plans for placing this terminal in a location which, because of the density of its occupation by all types of industrial establishments, public utilities, and railway lines and yards, was exceedingly complex and required the careful solution of detailed problems of many kinds. In this position, he also supervised all the construction work, except such features as were in the hands of the architects and the mechanical engineers.

The crowning achievement of his career as an engineer was the construction of the St. Paul Union Passenger Station. Mr. Armstrong went to St. Paul in February, 1915, and assumed the duties of Chief Engineer. A plan had previously been devised and adopted for this depot, which involved changing the channel of the Mississippi River at a cost of about $15 000 000. In the summer of 1915, this plan was rejected by the Army Engineer, and, therefore, had to be abandoned. Mr. Armstrong thereupon commenced the preparation of a new plan, which did not involve any change in the river, and which, it became apparent later, possessed many advantages over the original plan. It was adopted in the spring of 1917 and from that time until the day of his death, Mr. Armstrong was actively engaged in carrying this plan to completion. The work would have been completed, except for delays occasioned by the World War. Although he did not live to see the work more than about half finished, the plans for the entire structure had been thoroughly developed and can readily be executed by his successors. Despite the discouragement these delays must have occasioned him, his devotion to his work never flagged.

Mr. Armstrong was married in 1889 to Miss Emma E. Moody, who, with their daughter, Alice A., survives him.

He was deeply interested in engineering education and made it a practice to return to his Alma Mater to deliver addresses and lectures to the engineering students. He was actively engaged in furthering the interests of engineering societies, and was President of the Western Society of Engineers in 1912. After going to St. Paul, he interested himself in, and was influential in the organization of, the Minnesota Federation of Architectural and Engineering Societies, and labored unceasingly for its success.

Mr. Armstrong also took an active interest in civic affairs, and during his residence in Chicago was a member of a Commission appointed by Governor Deneen in 1914 to revise and codify the Building Laws of the State of Illinois. The recommendations of this Commission were subsequently adopted and became part of the law of that State. Recognition of his services was made by the Mayor of St. Paul at a public gathering, when he paid a fitting tribute

to Mr. Armstrong both as the builder of the Union Depot and as a civic worker.

Those who were closely associated with him speak of him only with praise. Contractors acclaim his fairness; friends extol his Christian ideals and conduct, his mild, unassuming manner, and his practical judgment. His younger professional associates acknowledge their indebtedness to him as an instructor and adviser. By his death, the Engineering Profession has lost a wise and discerning counselor in matters of professional organization and activity and in the relations of the engineer to society.

Mr. Armstrong was elected a Member of the American Society of Civil Engineers on June 1, 1909.

EDWARD THOMAS FLAHERTY, M. Am. Soc. C. E.*

DIED APRIL 1, 1923.

Edward Thomas Flaherty was born in Stockton, Calif., on February 17, 1877. He was graduated from the Stockton High School in 1885 and was accredited to the University of California or to Stanford University.

Mr. Flaherty, however, immediately entered active work. Beginning as Timekeeper and Rodman with the Alameda and San Joaquin Railroad Company (now the Western Pacific Railroad Company), Mr. Flaherty was next engaged with the Southern Pacific Company and, later, with the Atchison, Topeka and Santa Fe Railway Company, in which service he rose through the grades of Inspector, Transitman, and Division Engineer, to Assistant to the Chief Engineer. He was connected with much of the construction work on the Coast Lines of the Atchison, Topeka and Santa Fe Railway Company and with their water-front improvements on San Francisco Bay.

Leaving railroad work in 1906, Mr. Flaherty entered the employ of the Atlantic, Gulf and Pacific Company, as Engineer and Superintendent of Construction. In this capacity, he had charge of river protection, bridge and pier construction, dredging, and heavy foundation work, and it is interesting to note that, in 1909, while with this Company, he designed the first reinforced concrete pile used for pier construction in Southern California.

In order to perfect his knowledge of the use of concrete in hydraulic construction, Mr. Flaherty went to Colorado, for the Reinforced Concrete Pipe Company, where he designed and installed a number of concrete siphons, the first to be successfully built and operated in Colorado. These siphons which were constructed for the Central Colorado Power Company, ranged in length from 200 to 1 100 ft. and were operated under heads of from 50 to 110 ft.

Returning to California in 1910, Mr. Flaherty opened an office as Designing and Consulting Engineer, and his record of achievement since that date is a noteworthy one, especially for the magnificent structures of reinforced concrete and steel—principally bridges and buildings—with which he was connected as Structural or Consulting Engineer. These structures the value of

* Memoir prepared by S. A. Jubb, M. Am. Soc. C. E., and M. C. Halsey, Assoc. M. Am. Soc. C. E.

which is estimated at several million dollars, included fifteen or twenty bridges of note, of which two were for the City of Glendale, two for Riverside County, one for Santa Ana County, one for the City of Stockton, and one for San Luis Obispo County, California. The long list of buildings with which his name is connected include the five-story building for the Ford Motor Company, the ten-story Knickerbocker Building, the thirteen-story Corporation Building, the Trinity Auditorium Building, the six-story Western Electric Building, the five-story Aristo Building, the five-story Hosfield Building, the four-story Palmer Building, the eight-story Wilshire Fireproof Storage Building, the Premier Fireproof Storage Building, the Hollywood Auto Transfer and Fireproof Storage Building, two Bekins Fireproof Storage warehouses, and the three-story Western Lithograph Building, all in Los Angeles, Calif. His re-design of the engineering plans for the Forum Theatre is estimated to have saved the owners approximately $50 000. There is also included in the list of structures designed by Mr. Flaherty the Lick Dome Pier, at Ocean Park, Calif., and the Auditorium Pier for the City of Santa Monica, as well as the reinforced concrete elevator tower located in the Hollywood Foothills.

Beginning at the bottom, Mr. Flaherty rose, through his individual efforts and hard work, to a commanding position in his profession. He specialized in the design of warehouses and furniture depositories, and many structures of this character in Los Angeles, San Francisco, and other points in California, are included in his work. He was much interested in advancing the Engineering Profession, and was instrumental in having the Building Law of the City of Los Angeles changed to permit engineers to sign plans for buildings. He also wrote the ordinance for reinforced concrete flat-slab construction contained in the Building Code of Los Angeles.

He was married to Helene J. MacMurphy and, besides his widow, he leaves a daughter, Marian Eloise. Having had considerable business experience, Mrs. Flaherty was of great assistance to her husband, especially in the last few years of his life, and will continue his practice.

Mr. Flaherty was elected an Associate Member of the American Society of Civil Engineers on February 28, 1911, and a Member on October 11, 1920.

WILLIAM HENRY JAQUES, M. Am. Soc. C. E.*

DIED NOVEMBER 23, 1916.

William Henry Jaques was born in Philadelphia, Pa., on December 24, 1848. His ancestry was chiefly Huguenot, the early families having settled near Perth Amboy, N. J., and Newburyport, Mass.

His early education was obtained in the public schools of New Jersey, and, at the age of fifteen, he entered the U. S. Naval Academy at Annapolis, Md., from which he was graduated with honor in June, 1867. He immediately entered into active service and was promoted to the grades of Ensign in 1868, Master 1870, and Lieutenant in 1871. While in the Navy, he served on the

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

Constitution, the *Marblehead*, *Canandaigua*, *St. Louis*, and *Quinnebaug*, and also on the monitors, *Saugus* and *Manhattan*. He also served as Aide to the President, the Secretary of the Navy, and the Commandant of the Brooklyn Navy Yard.

From 1870 to 1874, Captain Jaques was Assistant with the United States Coast Survey, in charge of triangulation, surveys, chart-making, deep-sea research, and astronomical work; and from 1874 to 1878, he assisted the New York Board of Education in technical work. Subsequently, he was engaged for a year or more as Assistant Inspector of Ordnance, and, from 1883 to 1885, he acted as Secretary of the United States Gun Foundry Board, also serving during this time as a member of the Board for the Improvement of the Washington Branch of the Potomac River. In 1886 and 1887, he was Secretary of the Senate Committee on Ordnance and Warships.

In 1887, Captain Jaques became Superintendent for the Bethlehem Iron Company, in charge of the design, construction, and adaptation of machinery for the manufacture of ordnance and armor-plate, which position he retained until 1894. He then became associated with Mr. Horace See in general consulting work in connection with the manufacture and treatment of guns, armor, shafting, etc.

Captain Jaques introduced into the United States the system of fluid compression and hydraulic forging of heavy masses of steel and was the inventor of improvements in the manufacture of heavy ordnance and armor. He also originated the idea of the use of nickel in steel, and was associated with Captain John Ericsson in the development of submarine artillery. He had a large share in bringing the manufacture of ordnance and armor-plate in the United States to a high standard of excellence, and the later developments of the Bethlehem Works are in a great measure due to him.

In 1898, owing to failure of his health, Captain Jaques was obliged to give up active work on the design of submarine boats in which he was associated, as President, with the Holland Submarine Boat Company. He subsequently engaged in private practice as a Consulting Engineer.

Captain Jaques was the author of several books on ordnance and allied subjects. He was presented with the Whitworth Medal for his metallurgical work, and was decorated by the Emperor of Japan with the Order of the Rising Sun on account of his services to Japan just previous to the war with China.

Captain Jaques was a member of the following Societies: Society of Naval Architects and Marine Engineers, Russian Geographical Society, Navy Relief Society, American Association for the Advancement of Science, American Institute of Mining Engineers, Association Technique Maritime (France), Atlantic Deeper Waterways Association, Boston Society of Civil Engineers, Hooker Association of Massachusetts, International Association of Navigation Congresses, Iron and Steel Institute (Great Britain), Navy League of the United States, National Geographic Society, New Hampshire Historical Society, American Society of Mechanical Engineers, Institution of Mechanical Engineers (Great Britain), Military Service Institution, Naval History Society, United States Naval Institute, and the Institution of Naval Architects (Great Britain), and a Fellow of the American Geographical Society.

He was a Companion of the Military Order of the Loyal Legion and of the Naval Order of the United States.

Captain Jaques was married on June 20, 1889, to Elizabeth, daughter of Senator John P. Hale of New Hampshire, and, again, in 1897, to Mary Adele, daughter of the Rev. Henry Genever, Vicar of Barbadoes, British West Indies, who, with two sons born of the second marriage, William Henry, Jr., and David Rittenhouse, survived him.

Captain Jaques was elected a Member of the American Society of Civil Engineers on July 2, 1890.

ERASMUS DARWIN LEAVITT, M. Am. Soc. C. E.*

DIED MARCH 11, 1916.

Erasmus Darwin Leavitt, the son of Erasmus Darwin and Almira (Fay) Leavitt, was born at Lowell, Mass., on October 27, 1836. He received his education at the public schools of Lowell.

In 1852, he entered the machine shop of the Lowell Manufacturing Company, where he served as an Apprentice for three years. He afterward worked for one year with the firm of Corliss and Nightingale. In 1858, he was employed by Mr. Harrison Loring at the City Point Works, South Boston, Mass., as Assistant Foreman, and had charge of the construction of the engine of the U. S. S. *Hartford.*

From 1859 to 1861, Mr. Leavitt was Chief Draftsman for Thurston, Gardiner and Company, steam-engine builders, of Providence, R. I., resigning at the beginning of the Civil War to enter the U. S. Navy. He served through the War of the Rebellion, having been stationed on the gunboat *Sagamore,* of the Eastern Gulf Squadron, for two years, and was then transferred to construction duty at Baltimore, Md., Boston, Mass., and Brooklyn, N. Y.

In 1865, Mr. Leavitt was sent by the U. S. Government to the Naval Academy at Annapolis, Md., as Instructor in Steam Engineering. In 1867, he resigned this position to resume private practice as a Mechanical Engineer, specializing in pumping and mining machinery.

Mr. Leavitt's ability first became recognized in connection with the installation of the pumping engines at Lynn and Lawrence, Mass., both of which were tested by some of the best engineers in the United States. The results obtained from these tests were highly satisfactory and of great interest to the Profession.

He acted as Consulting Engineer of the Calumet and Hecla Mining Company from 1874 to 1904, during which period he designed and superintended the building of the enormous plant at Calumet, Mich. He also served as Consulting Engineer for Henry R. Worthington, the Dickson Manufacturing Company, the Cities of Boston and Cambridge, and designed the pumping engine for the Louisville, Ky., Water-Works. He designed the high-duty pumping engines for the sewerage system of Boston, which are still doing efficient

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

service, and also acted as Advisory Engineer and designed machinery for the Bethlehem Steel Company and various South African mining companies.

Mr. Leavitt was of a modest and retiring nature, and his life was one of close application to his work. He occupied a leading place among engineers and his ability was recognized both at home and abroad. During his many trips to Europe, he received marked attention from members of the Engineering Profession there. He probably did more than any other engineer in establishing sound principles of design and in advocating the use of the best materials in construction, realizing that reliability of service was a most important consideration.

He found little time for outside interests, but served on the Boston and Cambridge Bridge Commission, when the Harvard Bridge was contemplated and built, and he was also greatly interested in the Cambridge Young Men's Christian Association. Mr. Leavitt was a striking example—perhaps among the last—of engineers who without what is called technical training have achieved the highest distinction in the ranks of Mechanical Engineering and have left a memory which marks almost an epoch in the practice of steam engineering.

He died on March 11, 1916, at Cambridge, Mass., where he had lived for many years. He was married on June 5, 1867, to Miss Annie Elizabeth Pettit, the daughter of William Pettit, of Philadelphia, Pa. His wife died in 1889, and he was survived by his daughters, Mrs. Walter Wessellhoeft, Miss Margaret A. Leavitt, and Mrs. Paul A. H. van Daell.

In 1884, Mr. Leavitt received the Honorary Degree of Doctor of Engineering from Stevens Institute of Technology, Hoboken, N. J. He was an Honorary Member of the American Society of Naval Engineers, Boston Society of Civil Engineers, British Association for the Advancement of Science, New England Water Works Association, and the American Society of Mechanical Engineers; a Member of the American Institute of Mining Engineers, Institution of Civil Engineers, and the Institution of Mechanical Engineers; a Life Member of the Franklin Institute; and a Fellow of the American Academy of Arts and Sciences.

Mr. Leavitt was elected a Member of the American Society of Civil Engineers on July 2, 1873.

HENRY MANLEY, M. Am. Soc. C. E.*

DIED OCTOBER 28, 1919.

Henry Manley, the son of Salmon and Iza Annette (Howard) Manley, was born at North Bridgewater (now Brockton), Mass., on August 31, 1841. He came of old New England stock, having been descended from *Mayflower* Pilgrims.

Mr. Manley's early education was received at the "District School". Later, he spent a year at Bridgewater Academy, after which he entered the State

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

Normal School, at Bridgewater, from which he was graduated in 1860. Following his graduation, he spent two years in teaching.

At the outbreak of the Civil War, he enlisted in Company K of the Third Massachusetts Volunteer Militia, with which he served from September 23, 1862, until he was mustered out on June 26, 1863. The hardships which he endured during this period having seriously affected his health, he took a long sea voyage to Cape Town, South Africa, where he remained for a year in the photographic business.

On his return to the United States, Mr. Manley taught school for a short time, but having decided to become a Civil Engineer, he entered the office of the late J. Herbert Shedd, M. Am. Soc. C. E., in 1866. While connected with Mr. Shedd's office, Mr. Manley went to Providence, R. I., on a preliminary survey for the water-works. During his stay there, he was sent, temporarily, to work for the City Engineer of Boston, Mass., and he was finally transferred to Boston permanently and remained in that service until his retirement, forty-two years later.

His earliest work of importance in this position was making surveys for supplying water to Roxbury, and Dorchester, Mass., both of which had been annexed to Boston, and, later, he made surveys for the extension of the water service to Deer Island.

Mr. Manley made the surveys for, and had charge of the construction of, the sea walls on Atlantic Avenue, from Broad Street to Commercial Street, at Eastern Avenue, in Boston, and also had charge of the construction of numerous wharves and bridges, among which were the Federal Street Bridge to South Boston, the Chelsea Street Bridge from East Boston to Chelsea, the bridge on the extension of Congress Street to South Boston, and the rebuilding of the Broadway Bridge across Fort Point Channel.

In 1877, Mr. Manley was assigned to the examination and repair of the bridges of the city, and continued in that work until 1891, when he was placed in charge of street improvements, including the preparation of plans and specifications. He held this latter position until his retirement in 1910.

From 1891 to 1913, Mr. Manley was an Examiner for the Civil Service Commission of Massachusetts for Inspectors and on Civil Engineers Grades from the establishment of that service in 1897 to 1913. During his connection with the city work, he found time for private practice in various parts of New England.

On November 6, 1867, he was married to Miss Susan Elizabeth Marshall, of North Bridgewater, Mass., who survived him by less than two months. They had three sons, Laurence B. Manley, M. Am. Soc. C. E., Henry Manley, Assoc. M. Am. Soc. C. E., and Howard T. Manley, who is also an Engineer.

Mr. Manley served as Treasurer of the Boston Society of Civil Engineers from 1880 to 1892, when he was elected President. He also served as a Director for two years immediately following his Presidency. He was for many years one of its most prominent members, and was the very life of its meetings especially at its annual dinners and other functions which were never complete without him. He presented several papers before the Boston Society,

the most interesting and important of which was that on "Rapid Transit in Boston",* in 1889.

He was also a member of the Massachusetts Highway Association and its President in 1896, and Honorary Member of the Connecticut Highway Association. His outside interests were many, as he was active in the Unitarian Church and the Unitarian Club of Boston, as well as a member of the Highland Club of West Roxbury and the Boston City Club, and, in later years, of the New England Genealogical Society. He was also prominent as a member of Charles W. Carroll Post No. 144, G. A. R., at Dedham, Mass.

Mr. Manley was a man of strong social instincts and his inexhaustible fund of humor and his Yankee "horse sense" furnish the key to his character. He was always a delightful companion and possessed a wide circle of friends among whom he was admired for his attainments, respected for his high character, and loved for his humanity. Perhaps more than most men, he is entitled to be "written down" with Abou Ben Adhem "as one who loves his fellow men."

Mr. Manley was elected a Member of the American Society of Civil Engineers on June 2, 1880, and served as a Director of the Society from 1898 to 1900.

HENRY HERMAN MARDEN, JR., M. Am. Soc. C. E.†

DIED MAY 18, 1910.

Henry Herman Marden, Jr., was born at Kittery, Me., on September 18, 1864. His education was acquired chiefly in the public schools of Boston, Mass., and at Boston College.

At the age of eighteen, Mr. Marden went to work as Rodman in the City Surveyer's Office, in Boston, under the Chief Surveyor, Mr. Thomas W. Davis. He was soon transferred from this office to that of the Massachusetts Drainage Commission, then being organized, where he remained for two years. He then entered the Boston Main Drainage Office, leaving there, in 1886, to engage in construction work on a bridge at Greenville, Conn., and engineering work at Norwich, Conn., under the late J. Herbert Shedd, M. Am. Soc. C. E. Mr. Marden became Assistant Engineer on this work, on which he remained until 1890, having been also employed, in the meantime, on the completion of a high masonry dam at Bridgeport, Conn., on the construction of a pulp mill at Livermore Falls, Me., on reservoir construction at Portland, Me., and on the completion of improvements to the water supply of Wellington, Kans.

In 1890, Mr. Marden became Assistant Engineer, under Howard A. Carson, Hon. M. Am. Soc. C. E., on the Metropolitan Sewerage System of Boston, in general charge of inspection of contract work. In this position, he was employed on the design and construction of the siphon for Belle Isle Inlet at East Boston, on the Chelsea Creek Siphon (compressed air), on a similar and larger tunnel at the Mystic River Crossing, at Charlestown, Mass., and on the pumping stations at the latter place and at East Boston.

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* *Journal*, Assoc. of Eng. Societies, Vol. VIII (1889), p. 280.

† Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

In 1895, Mr. Marden was appointed Assistant Engineer on the Boston Transit Commission on the construction of subways for Boston. He was first engaged on studies in the office, but was afterward placed in charge of the construction of the tunnel in Tremont Street, which the contractor had been forced to abandon. This work which was completed by Mr. Marden with day labor, involved considerable ingenuity and was finished under vexatious conditions. On its completion, Mr. Marden worked for a time with the H. Nawn Contracting Company, following which he was associated, for two years, with Mr. F. A. McClure, then City Engineer of Worcester, Mass., on the construction of the separate system of sewerage for that city.

In 1900, he became Superintendent of Construction for the Michigan Lake Superior Power Company. He was employed subsequently on the Sault Ste. Marie Canal, which position he resigned to go with the United Engineering and Contracting Company, of New York City. While in the employ of this Company, he was engaged on the approaches to the Williamsburgh Bridge, New York City, Vernon Avenue Bridge, Long Island City, N. Y., and on the Pennsylvania Tunnels, at Thirty-third Street, New York City. He also supervised other responsible work for the Company in America and Europe, remaining in its employ until his death, which occurred at the Bremerhaven Hospital, Bremen, Germany, from pneumonia contracted while he was on his way to Moscow, Russia, on work for the Russian Government.

Mr. Marden was a man of indefatigable activity and of great resource, and he possessed that faculty, lacked by so many engineers, confidence in his own power to succeed.

He was married on June 8, 1892, to Miss Mary L. Stratton, who, with four children, Katherine, Henry Herman, Jr., Marie, and Madeline E., survives him.

Mr. Marden was elected a Member of the American Society of Civil Engineers on May 1, 1901.

FRED STARK PEARSON, M. Am. Soc. C. E.*

DIED MAY 7, 1915.

Fred Stark Pearson was born at Lowell, Mass., on July 3, 1861. His father, Ambrose Pearson, a civil engineer engaged in railroad work, died when he was fifteen years of age.

In 1877, the family having moved to Medford, Mass., Mr. Pearson became Station Agent at Medford Hillside, on the Boston and Lowell Railroad. Here, he attracted the attention of some of the teachers at Tufts College and as a result he entered this institution in 1879. In 1880, he transferred to the Massachusetts Institute of Technology for a course in Chemistry, but returned in 1881 to Tufts College. At this time, although not yet twenty-one, he served as Postmaster at College Hill.

Mr. Pearson was graduated from Tufts College in 1883, and was immediately given a three years' appointment as Walker Instructor in Mathematics,

* Memoir prepared by Frederic I. Winslow, M. Am. Soc. C. E.

at the same time pursuing further studies, as well as investigations for various commercial interests. He afterward turned his attention to mining engineering in the United States and Brazil, and to the organization of electric lighting companies in Somerville and Woburn, Mass., and in Halifax, N. S., Canada.

In 1888, Mr. Henry M. Whitney, then President of the West End Street Railway of Boston, Mass., invited Mr. Pearson to assume charge of the development of the electric transportation service of that city, at the exceptional salary of $2 500. During the two years that Mr. Pearson remained in this position, he increased the size of the generators at the main power plant from 120 to 500 h. p., a size then considered so large that the Westinghouse Company refused to bid on the work, and the Thomson-Houston Company, after some demur and with a modification of the contract, carried it out. Years afterward, the late George Westinghouse, M. Am. Soc. C. E., referred to these specifications as marking an epoch in the development of the dynamo.

Subsequently, when Mr. Pearson prepared specifications for six large triple-expansion engines, he found a similar difficulty in inviting bids, the manufacturers declining, as they said, "to experiment with new and freak forms of pumping machinery." Later, however, the head of the firm which finally built these engines, the E. P. Allis Company, of Milwaukee, Wis., offered Mr. Pearson the position of Chief Engineer of that Company at his own salary.

In 1894, Mr. Pearson was called to New York City as Chief Engineer of the Metropolitan Street Railways, also serving as Consulting Engineer for other railways, notably those of Toronto, Ont., Montreal, Que., St. John, N. B., and Halifax, N. S., in Canada; Syracuse, N. Y., Providence, R. I., Staten Island, N. Y., Newark and Asbury Park, N. J., the Metropolitan Elevated Railroad, Chicago, Ill., and the Atlantic Coast Railways, in the United States; Havana, Cuba; and the Birmingham Tramways Company, and the Liverpool Street Railroad, in England. He also served as Consulting Engineer for the power transmission line from Chambley to Montreal, as well as for power transmission at Kingston, Jamaica.

He was consultant on the reconstruction of shipping piers at Sydney and at Louisburg, Cape Breton, Canada; the installation of coal-handling apparatus at Montreal; the construction of the Sydney and Louisburg Railroad, and the construction and equipment of the Railroad Company's mines at Cape Breton. Mr. Pearson also served as Chief Engineer for the Dominion Coal Company, refusing, later, the Presidency of that Company.

Becoming interested in water-power problems, he undertook the development of the Rio Tiete, Brazil, for furnishing power to the City of São Paulo; he was then called to Mexico, to engage in a similar development in the State of Puebla at the Falls of Necaxa, his investigations resulting in the organization of the Mexican Light and Power Company, and the construction of a plant with a capacity of 40 000 h. p. which, when fully developed, transmitted between 100 000 and 200 000 h. p. to the City of Mexico, 95 miles distant. Still later, he had charge of a similar development of almost exactly the

same magnitude at Niagara Falls. Another undertaking was the 25 000 h. p. plant at Lac de Bonnet, on the Winnipeg River, for the City of Winnipeg, Man., Canada.

His next great enterprise was the development of a power plant of about 40 000 h. p. for the City of Rio de Janeiro, Brazil. His last important work was the development of the Ebro River, in Spain, for the general use of the City of Barcelona. This plant was nearly completed when the World War began and compelled a general halt in the work.

Mr. Pearson was married on January 7, 1887, to Miss Mabel Ward, daughter of the prominent contractor, William H. Ward. Mrs. Pearson shared the tragic fate of her husband who was drowned by the sinking of the *Lusitania* on May 7, 1915. They were survived by two sons, Ward Edgerly and Fred, both engineers, and by one daughter, Natalie, the wife of Reginald Nicholson, Manager of the *London Times*.

Mr. Pearson was a member of the following technical societies and clubs: American Institute of Electrical Engineers; American Society of Mechanical Engineers; American Society of Naval Engineers; Institution of Civil Engineers; Boston Society of Civil Engineers; the American Institute of Mining Engineers; the Mining Society of Nova Scotia; Massachusetts Charitable Mechanics Association; University Club, Boston; Engineers Club, New York; Jockey Club, City of Mexico; St. James Club, Montreal; Toronto Club, Toronto; and many others. He was made a Trustee of Tufts College in 1900, and was given the degree of Doctor of Science, in 1900, and that of Doctor of Laws, in 1905.

From his earliest days, Mr. Pearson was a man of tireless energy, and was fond of work for the sake of the result. He might fittingly be called, as was Daniel Webster, "a steam-engine in trousers". Every subject that he touched he seemed to absorb and master as though he had a special aptitude for every science. His versatility of intellect was marked by all who knew him. He possessed a constructive and creative imagination without which he could never have achieved the enormous works he left, involving, as they did, great originality and prompt comprehension of complicated situations. He possessed the quality of endearing himself to his associates in business and was always willing to accept constructive suggestions. He was also fortunate in commanding the unswerving loyalty of his subordinates.

Mr. Pearson was elected a Member of the American Society of Civil Engineers on November 3, 1897.

PRESTON KING YATES, M. Am. Soc. C. E.*

DIED APRIL 22, 1921.

Preston King Yates was born at Canajoharie, N. Y., on December 17, 1856. He received his technical education at Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1880 with the degree of Civil Engineer.

* Memoir compiled from information on file at the Headquarters of the Society.

In 1880, Mr. Yates was employed as Assistant Engineer in charge of construction on the New York, West Shore, and Buffalo Railroad. From 1882 to 1883, he was Location Engineer in charge on the South Pennsylvania Railroad, and, during the following two years, served as Location Engineer in charge on the Fort Plain, Richfield, and Cooperstown Railroad.

From 1885 to 1886, and during the spring of 1887, he was engaged as Assistant Engineer on the construction of the New Croton Aqueduct. Having completed this work, he went to Natchez, Miss., where he was appointed Chief Engineer of the Natchez Water and Sewer Company and also City Engineer of Natchez. In this capacity, he designed and completed the water and sewer system for the city. When this work was finished in 1890, he was engaged as an Assistant Engineer with the Subway Commission of New York City.

On leaving the service of the Subway Commission, Mr. Yates became Assistant Engineer to the late Alfred P. Boller, M. Am. Soc. C. E., with whom he remained until December, 1892. The 155th Street Viaduct, and the new Central Bridge across the Harlem River, in New York City, were among the works on which he was engaged during this period.

From 1900 to 1904, he served as First Assistant Engineer of the Lake Shore and Michigan Southern Railroad, with headquarters at Cleveland, Ohio. During the five years following, he was engaged in general contracting and engineering construction.

In 1911, Mr. Yates entered private practice as a Consulting Engineer. He was an expert on stone-crushing and ore-crushing plants, and as Consulting Engineer, he designed and erected the Tomkins Cove Stone Crushing Plant at Tomkins Cove, N. Y.; the plant of the New Haven Trap Rock Company, at North Branford, Conn.; the plant of the Texas Trap Rock Company, at Thurber, Tex., and completed the design of the plant for the Georgia State Highway Commission.

During the World War, Mr. Yates acted for the Government as a Consulting Engineer on the Muscles Shoals Plant of the Air Nitrates Company.

Mr. Yates was elected a Junior of the American Society of Civil Engineers on June 6, 1883, and a Member on April 5, 1893.

KINGSLEY PARKHURST ARNOLL, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 1, 1923.

Kingsley Parkhurst Arnoll, the son of Robert G. and Annie S. Arnoll, was born in Bridgewater, Vt., on July 19, 1885. He was educated in the public schools of Rutland, Vt., having been graduated from the High School in 1902. From that time until 1906, he was employed by the Rutland Railroad Company.

In 1906, Mr. Arnoll went to New York City, where he remained, in the employ of the Engineering Department of the New York Central and Hudson River Railroad Company, until 1913. He was Chief Clerk to the Office Engi-

* Memoir prepared by Roy F. Bessey and Lewis M. Hammond, Associate Members, Am. Soc. C. E.

neer from 1908 to 1911, when he went into engineering work and was engaged as Transitman and Draftsman until 1913. During this period, Mr. Arnoll successfully completed the five-year evening course at Cooper Union, having been graduated with the degree of B. S. in 1912, and an evening post-graduate course in advanced structural design at the Brooklyn Polytechnic Institute.

From 1913 to 1916, he was employed by the Terminal Construction and Municipal Engineering Divisions of the Panama Canal on the design of many important structures at the Pacific Terminal of the Canal, and on municipal improvements.

In 1916, he entered the employ of the Bureau of Yards and Docks of the United States Navy Department where he remained, first, as leading Draftsman and, later, as Assistant Project Manager, until the time of his death. As leading Draftsman, Mr. Arnoll had charge of the preparation of plans and specifications for many large and important Navy Yard structures, principally water-front work and industrial buildings. As Assistant Project Manager, during the World War, he performed executive work in connection with the operations of the Shipbuilding and Storage Facilities Section of the Bureau. This work entailed the expenditure of approximately $60 000 000 and included improvements at all the Navy Yards.

Mr. Arnoll's character and reputation were above reproach, and he was held in high esteem by all his business associates. He had a comparatively small number of intimate friends, but to these he was a true friend in the highest sense of the word. By nature, he was quiet and studious, and a greater part of his spare time was spent in further study of the problems encountered in his work. Mr. Arnoll had been in failing health for about two years prior to his death, due in a large measure to overwork during the war. He was not married, and is survived by his parents, two brothers, and a sister.

Mr. Arnoll was elected an Associate Member of the American Society of Civil Engineers on April 16, 1918.

LOWELL GAYNOR KRIGBAUM, Assoc. M. Am. Soc. C. E.*

DIED MAY 10, 1921.

Lowell Gaynor Krigbaum, the only child of Henry S. and Helen M. Krigbaum, was born in Helena, Mont., on May 30, 1888. When he was five years of age, his parents moved to San Francisco, Calif., where he attended the public schools, having been graduated from the Lowell High School in 1906.

From 1906 to 1908, Mr. Krigbaum was connected with the Southern Pacific Railroad Company, as Rodman, Levelman, Draftsman, and Transitman on preliminary location and construction surveys for hydro-electric development. In 1908, he entered the College of Mines of the University of California, from which he was graduated in 1912 with the degree of B. S. During his college vacations, he continued his work with the Southern Pacific Railroad

* Memoir prepared from information on file at the Headquarters of the Society.

Company, and immediately after his graduation, he joined the Company's forces as Transitman on railroad construction.

In January, 1913, Mr. Krigbaum accepted a position as Draftsman and Chief of Field Party on the Solano Irrigated Farms, a 100 000-acre reclamation and irrigation project, and was engaged on the estimation for and design of canals and structures.

From August, 1913, to April, 1915, he served in the United States Engineer Department at Large, Third San Francisco District, as Chief of Survey Party on river surveys, and from April, 1915, to September, 1917, he was employed as a Junior Engineer with the U. S. Engineer Department and the California Débris Commission, at San Francisco, in charge of the regulation of hydraulic mining and gold dredging operations and of the design of restraining works and river improvement on the Yuba River. He inspected the project for the Hetch Hetchy Water Supply for San Francisco, and reported to Washington on the subject.

When the United States entered the World War, Mr. Krigbaum volunteered his services and entered the Officers' Training Camp at Vancouver, Wash., where he remained for three months. In December, 1917, he was sent overseas as a First Lieutenant, Engineer Corps, United States Army, and was assigned to the Water Supply Section of the Division of Construction and Forestry, at Tours, France. In October, 1918, he was promoted to the rank of Captain. His work at this time included the design of water supply and sewage disposal systems, and he also served as Executive Assistant to the Officer in charge of the Water Supply Section. He remained abroad until June, 1919, when he returned to the United States.

Arriving in San Francisco on July 7, 1919, Captain Krigbaum returned at once to the employ of the U. S. Engineer Department at Large and the California Débris Commission as Junior Engineer, in which capacity he was in charge of projective planning for river improvement works and the regulation of gold dredging operations. In addition to this position, he was associated with Messrs. A. O. Stewart and Winship on a mining and irrigation project, to which he gave all his spare time. He was just about to resign his position with the California Débris Commission to take charge of that work, when he was killed in an automobile accident on May 10, 1921, while returning from an inspection trip in connection with the project.

In September, 1917, Captain Krigbaum was married to Miss Jerome Gerhart, of Covina, Calif., who, with a son, survives him. He is also survived by his parents.

He was exceptionally honest and upright in character and was greatly loved and respected by all who knew him. Although quite young at the time of his death, Captain Krigbaum had shown himself exceedingly capable as an engineer, his executive ability and genius for handling men having been noted on many occasions. In his home life, he was a devoted son, husband, and father.

He was a member of the Presbyterian Church, a Mason, and also a member of the American Legion.

Captain Krigbaum was elected a Junior of the American Society of Civil Engineers on November 4, 1914, and an Associate Member on August 9, 1920.

CLARENCE IVAN LANTZ, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 4, 1921.

Clarence Ivan Lantz, the son of John F. Lantz, was born at Frosty Run, Pa., on November 12, 1882. His common school education was received at that place, and his college preparatory education at West Virginia University which, at that time, had a Preparatory Department. In 1900, he entered West Virginia University, where he studied Civil Engineering.

During the summer of 1903, Mr. Lantz was employed in the office of the Riter-Conley Manufacturing Company of Pittsburgh, Pa., detailing structural work for track elevation of the Pennsylvania Railroad at Chicago, Ill.

He was engaged, from 1904 to 1908, with The American Bridge Company, at Ambridge, Pa., detailing and checking drawings for bridges, mill buildings, mine structures, ore docks, and other steel work.

From 1908 to 1910, he was a member of the firm of Boughton and Lantz, Engineers, at Morgantown, W. Va., engaged in general engineering practice, specializing in bridge and building design and superintendence. For about a year following, Mr. Lantz devoted his attention to the construction of concrete bridges and acted as Engineer of Construction for various counties of West Virginia.

His next engagement was at Morgantown, W. Va., with the Buckhannon and Northern Railroad Company (afterward incorporated into the Pittsburgh and Lake Erie System), as Designing Engineer, reporting to S. D. Brady, M. Am. Soc. C. E., Chief Engineer. In this capacity, he designed all the bridges for this new road extending from Fairmont, W. Va., to the Pennsylvania State line.

Mr. Lantz then became interested in several Morgantown industries, among which was the Chrisman-Goodwin Foundry Company, of which he was Secretary, in charge of all engineering work. He served also, at Morgantown, as Bridge Engineer for the County Court of Monongalia County, West Virginia.

Mr. Lantz had always been financially interested in coal lands and, in 1914, began to concentrate his attention on the coal business, developing mines near Morgantown. In 1917, he was the leader of a group that organized the Rosedale Coal Company to operate mines in West Virginia. He was made General Manager of that Company as well as of the Mapletown Coal Company, a subsidiary operating in Pennsylvania. He was also Sales Manager of another subsidiary, the Blue Flame Fuel Company, organized as a sales corporation. His relations with these Companies continued until his death.

On June 23, 1904, Mr. Lantz was married to Miss Anna B. Hennen, who, with their only child, a daughter, survives him.

Mr. Lantz made friends easily and retained them. While at West Virginia University, he became a member of Phi Kappa Sigma. He attained high

* Memoir prepared by W. H. Boughton, Assoc. M. Am. Soc. C. E.

offices in the Masonic Fraternity, having been a member in the York and Scottish Rite bodies and the Knights Templars. He was a member and an officer of the Methodist Episcopal Church, and also a member of the Kiwanis Club.

Mr. Lantz was elected an Associate Member of the American Society of Civil Engineers on October 1, 1913.

CHARLES WILSON ROSS, Assoc. Am. Soc. C. E.*

DIED APRIL 11, 1916.

Charles Wilson Ross, the son of William J. Ross, was born in West Boylston, Mass., on March 8, 1849. His father was a builder, who passed much of his time on a farm and, incidentally, invented many useful and labor-saving appliances for industrial and agricultural uses.

By attending the district school of his native town, and by persistent study in later years, Mr. Ross acquired a sound business education. He worked for his father, in the shop and on the farm, until 1869, when he went to Newton, Mass., and established his permanent home.

Mr. Ross was appointed Assistant Superintendent of the Newton Cemetery, which position he held for twenty-one years. During this time, he achieved much success in landscape gardening and in laying out private estates. His genius with plant life was so marked that his cousin, Luther Burbank, repeatedly urged him to go to California in order that they might work together.

On December 8, 1890, he was elected Superintendent of Streets of Newton, and, in 1898, when all the public works were placed under one Commissioner, Mr. Ross was appointed to that office with the title of Street Commissioner. This office he filled until April 15, 1914, when he resigned, to accept the position of Superintendent of Newton Cemetery, which he held until his death.

In 1898, Mr. Ross was appointed a member of the Massachusetts Highway Commission by Governor Walcott. At the end of two years of efficient service, he was obliged to resign because of the demands of his work at Newton.

Mr. Ross had executive ability, was honest and fair in all his dealings, and was of a kindly and sympathetic disposition, yet exacting full service from all employees. His economy was of the type that used capitalized maintenance as the limit of cost. He was a progressive engineer and one of the first to take into account the character of the old roadbed and the kind of traffic, in determining the new thickness of the road metal. Under his administration, the road system of Newton achieved a National reputation, and his professional advice was much sought.

Mr. Ross was a Charter Member of the Massachusetts Highway Association (having served as its President), of the Massachusetts Horticultural

* Memoir prepared by W. E. McClintock and Frederic I. Winslow, Members, Am. Soc. C. E.

Society, American Park Association, National and New England Cemetery Associations, Boston Society of Civil Engineers, and the Dalhousie Lodge of F. and A. M., of Newton. He was affiliated with the Baptist Church.

He was married to Miss Ella E. Gould, who, with five children, H. Wilson Ross, George W. Ross, Mrs. Edward E. Lothrop, Mrs. Arthur Hodges, and Mrs. Edward E. Savory, survived him. His home life was ideal and an inspiration to those friends who were admitted within its charming circle.

Mr. Ross was elected an Associate of the American Society of Civil Engineers on February 28, 1911.

VOL. XLIX

SUBJECT INDEX

Addresses
Address at the Annual Convention in Chicago, Illinois, July 11, 1923. Charles F. Loweth. 1363.

Aerial Photography
See Photography, Aerial.

Aerial Surveying
See Surveying, Aerial.

Aeronautic Maps
Aerial photography and map making, with special reference to water power surveys, 117, 119, 120, 984, 987.

American Society of Civil Engineers
Address at the Annual Convention in Chicago, Illinois, July 11, 1923. Charles F. Loweth. 1363.
Research activities of the ——. 512.

Aqueducts
"Engineering Geology of the Catskill Water Supply." Discussion: William W. Brush, Thomas H. Wiggin, Joel D. Justin, 123; James F. Kemp, Thaddeus Merriman, Charles P. Berkey, James F. Sanborn, 568.

Arches
"Improved Type of Multiple-Arch Dams." Fred A. Noetzli. 1065. Discussion: B. F. Jakobsen, Victor H. Cochrane, William Cain, 1767; H. deB. Parsons, Herman Schorer, 1924; H. J. F. Gourley, Alfred D. Flinn, 2067.
"Reactions for a Particular Type of Unsymmetrical Arch." Carl B. Andrews. 1685. Discussion: Victor H. Cochrane, J. Charles Rathbun, 2082.
"Stresses in Multiple-Arch Dams." B. F. Jakobsen. 1093. Discussion: William Cain, 1785; Lars R. Jorgensen, 1927; Fred A. Noetzli, 2069.
"The Comparison of Concrete Groined—as an Aid in Their Design." Discussion: Thomas H. Wiggin, 251; Philip O. Macqueen, 575.

Beams
"Tentative Specifications for Steel Highway Bridge Superstructure." Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.

Borers
Marine borers. 138.

Borings
—— at Ashokan Reservoir of the Catskill Water Supply System. 1744.
"Engineering Geology of the Catskill Water Supply." Discussion: William W. Brush, Thomas H. Wiggin, Joel D. Justin, 123; James F. Kemp, Thaddeus Merriman, Charles P. Berkey, James F. Sanborn, 568.

Breakwaters
"Ocean Beach Esplanade, San Francisco, Calif." M. M. O'Shaughnessy. 1846.

Bridges
"Final Report on Specifications for Design and Construction of Steel Railway Bridge Superstructure." Submitted by the Special Committee on Specifications for Bridge Design and Construction. 53.
"Locomotive Loadings for Railway ——." Discussion: Charles Evan Fowler, C. F. Loweth, Ralph Modjeski, C. A. P. Turner, 131; D. B. Steinman, 952.
"Progress Report of the Special Committee on Impact in Highway ——." 457.
"Proposed Loading for Highway ——." Harold D. Hussey. 1031. Discussion: J. A. L. Waddell, Otis E. Hovey, D. B. Steinman, R. de Charms, Samuel J. Ott, Lewis E. Moore, Glenn B. Woodruff, 1901.
"Tentative Specifications for Steel Highway Bridge Superstructure." Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.

Canals
Inner Harbor Navigation Canal at New Orleans, La. 1173, 1179.

Cast Iron
—— columns. 175, 216, 238.

Cement
Tests of sand briquettes. 137.
"The Disintegration of —— in Sea Water." William G. Atwood and A. A. Johnson. 1038. Discussion: Nathan C. Johnson, W. M. Black, Alfred D. Flinn, Benjamin A. Howes, Thomas H. Wiggin, Robert Ridgway, Richard H. Gaines, J. R. Slattery, Kenneth Allen, John Charles Riedel, Charles Rufus Harte, Chandler Davis, Alfred H. White, 1647; B. Jeanneret, A. H. Rhett, I. Hiroi, Theodore Belzner, 1762; E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, Jasper O. Draffin, 1914; Charles A. Newhall, Eduardo de Castro, R. Feret, 2060.

City Planning
"——." Technical Papers Presented at the Annual Meeting, January 19, 1923. Nelson P. Lewis, 554; George H. Norton, 561; Linn White, 702; Morris Knowles, 713. Discussion: William T. Lyle, E. M. Walker, Charles W. Leavitt, Frederic A. Delano, Charles N. Lowrie, M. W. Weir, E. A. Fisher, Rudolph Hering, Harold M. Lewis, George P. Hemstreet, George A. Soper, Nelson P. Lewis, Harold A. Caparn, 753.

Columns
"Reinforced Concrete ——." John Tucker, Jr., 165. Discussion: Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, 787; H. V. Spurr, Austin H. Reeves, 999.

Commerce
"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr.; D. J. Brumley, Frederick E. Morrow, Robert H. Ford, J. D'Esposito,

Rufus W. Putnam, William L. R. Haines, 1405. Discussion: Clement C. Williams, 1799; B. F. Jakobsen, William T. Lyle, 1936.

Concrete
Test of —— in sea water. 137.
"The Comparison of —— Groined Arches as an Aid in Their Design." Discussion: Thomas H. Wiggin, 251; Philip O. Macqueen, 575.
"The Disintegration of Cement in Sea Water." William G. Atwood and A. A. Johnson. 1038. Discussion: Nathan C. Johnson, W. M. Black, Alfred D. Flinn, Benjamin A. Howes, Thomas H. Wiggin, Robert Ridgway, Richard H. Gaines, J. R. Slattery, Kenneth Allen, John Charles Riedel, Charles Rufus Harte, Chandler Davis, Alfred H. White, 1647; B. Jeanneret, A. H. Rhett, I. Hiroi, Theodore Belzner, 1762; E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, Jasper O. Draffin, 1914; Charles A. Newhall, Eduardo de Castro, R. Feret, 2060.
See also Reinforced Concrete.

Conduits
Danger from pipes or —— passing through earth dams. 900.
"Elastic Stresses in the Rock Surrounding Pressure Tunnels." Charles P. Dunn. 661. Discussion: B. F. Jakobsen, Jacob Feld, William Cain, 1264.

Costs of Work
Classification of —— of diversion on delta of the Colorado River. 690.
Construction costs of Ocean Beach Esplanade, San Francisco, Calif. 1857.

Curves
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Dams
"Engineering Geology of the Catskill Water Supply." Discussion: William W. Brush, Thomas H. Wiggin, Joel D. Justin, 123; James F. Kemp, Thaddeus Merriman, Charles P. Berkey, James F. Sanborn, 568.
"Experiments with Models of the Gilboa Dam and Spillway." Discussion: B. F. Groat, Robert Fletcher, I. Gutmann, Robert F. Ewald, Thaddeus Merriman, 240; R. W. Gausmann, C. M. Madden, 566.
Failures and partial failures of earth ——. 1892.
"Improved Type of Multiple-Arch ——." Fred A. Noetzli. 1065. Discussion: B. F. Jakobsen, Victor H. Cochrane, William Cain, 1767; H. de B. Parsons, Herman Schorer, 1924; H. J. F. Gourley, Alfred D. Flinn, 2067.
"Some Notes on the Location and Construction of Locks and Movable —— on the Ohio River, with Particular Reference to Ohio River Dam No. 18." Discussion: William M. Hall, 771.
Statistics of successful earth ——. 913.
"Stresses in Multiple-Arch ——." B. F. Jakobsen. 1093. Discussion: William Cain, 1785; Lars R. Jorgensen, 1927; Fred A. Noetzli, 2069.
"Tentative Plan for the Construction of a 780-Foot Rock-Fill Dam on the Colorado River at Lees Ferry, Arizona." Discussion: E. C. La Rue. 942.
"The Design of Earth ——." Joel D. Justin. 856. Discussion: James F. Sanborn, Allen Hazen, E. G. Haines, A. H. Pratt, Thomas H. Wiggin, J. Albert Holmes, Gardner S. Williams, Charles H. Paul, H. B. Muckleston, H. J. F. Gourley, Charles Haydock, 1296; Caleb Mills Saville, Fred A. Noetzli, C. S. Jarvis, 1608; Thaddeus Merriman, John G. Hirsch, 1744; John E. Field, J. C. Stevens, Joel D. Justin, 1874.

Division of Engineering, National Research Council
See National Research Council.

Docks
Public —— at New Orleans, La., 1182.
See also Wharves.

Draft Tubes
"Comparative Tests on Experimental Draft-Tubes." 1813.

Earth Pressures
"Lateral Earth Pressure: The Accurate Experimental Determination of the Lateral Earth Pressure, Together with a Résumé of Previous Experiments." Jacob Feld. 603. Discussion: J. C. Meem, E. G. Haines, George Paaswell, D. B. Steinman, M. J. McPike, Edward Godfrey, G. M. Braune, Thomas H. Wiggin. 1007.

Education
"Engineering ——": Technical Papers Presented at a Joint Session with the Society for the Promotion of Engineering —— at the Annual Meeting January 17, 1923. Charles F. Scott, William G. Raymond, John L. Harrington, 491; Magnus W. Alexander, 698. Discussion: Robert Fletcher, F. B. Sanborn, J. K. Finch, George F. Swain, C. M. Spofford, Dugald C. Jackson, Theodore T. McCrosky, T. Chalkley Hatton, J. C. Ralston, Milo S. Ketchum, W. E. Wickenden, Charles F. Scott, Sydney Wilmot, H. R. Buck, A. B. McDaniel, R. L. Sackett, 729; Howard Constable, 997.

Electric Generators
"The Design of Structural Supports for Turbo-Generators." Edward H. Cameron. 37. Discussion: W. E. Belcher, Austin H. Reeves, F. A. Ericson, James H. Richardson, F. M. Beer, 274; George A. Orrok, 581; R. von Fabrice, 782; D. A. Allee, Edward H. Cameron, 993.

Electric Power
High voltage power transmission. 579.

Electrification of Railroads
See Railroads.

Engineering Education
See Education.

Engineering Foundation
——: Division of Engineering of National Research Council: Their origin, work, plans, needs. 518.

Engineering Research
See Research.

Engineering Societies
Address at the Annual Convertion in Chicago, Illinois, July 11, 1923. Charles F. Loweth, 1363.
Co-operation of National —— in engineering education. 506.
Engineering Foundation: Division of Engineering of National Research Council: Their origin, work, plans, needs. 518.

Erosion
"A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California." S. L. Rothery. 671. Discussion: Paul M. Entenmann, 1285.
"Ocean Beach Esplanade, San Francisco, Calif." M. M. O'Shaughnessy. 1846.
"The River and Harbor Problems of the Lower Mississippi": A Symposium.

Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, John Klorer, John F. Coleman, Samuel McCain Young, John R. Freeman, 1133; Elliott J. Dent, 1722; M. J. Sanders, Joseph E. Ransdell, 2021. Discussion: B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, C. E. Grunsky, 1250; John R. Freeman, W. G. Price, J. A. Ockerson, Sidney F. Lewis, C. W. Chapin, C. McD. Townsend, 1624; W. M. Black, H. B. Muckleston, H. C. Ripley, H. F. Dunham, 1792; Elliott J. Dent, 1929; Lewis M. Haupt, C. E. Grunsky, William T. Lyle, 2045.

Floods
Maximum recorded —— in American rivers. 1896.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, John Klorer, John F. Coleman, Samuel McCain Young, John R. Freeman, 1133; Elliott J. Dent, 1722; M. J. Sanders, Joseph E. Ransdell, 2021. Discussion: B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, C. E. Grunsky, 1250; John R. Freeman, W. G. Price, J. A. Ockerson, Sidney F. Lewis, C. W. Chapin, C. McD. Townsend, 1624; W. M. Black, H. B. Muckleston, H. C. Ripley, H. F. Dunham, 1792; Elliott J. Dent, 1929; Lewis M. Haupt, C. E. Grunsky, William T. Lyle, 2045.

Foundations
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for ——", etc. 1732.
"The Design of Structural Supports for Turbo-Generators." Edward H. Cameron. 37. Discussion: W. E. Belcher, Austin H. Reeves, F. A. Ericson, James H. Richardson, F. M. Beer, 274; George A. Orrok, 581; R. von Fabrice, 782; D. A. Allee, Edward H. Cameron, 993.

Freight
"Analysis of Cost of —— Service, Grand Trunk Railway Company of Canada." J. P. Newell. 3. Discussion: J. G. Sullivan, 991.
Modern —— terminal of the Pennsylvania Railroad system in Chicago, Illinois. 1585.
Modern rail and water terminals with particular reference to the situation at Chicago, Illinois. 1552.
Railroad —— terminal problem in St. Louis, Missouri. 1464.
Railroads—the arteries of commerce. 1417.
Some phases of present-day railroad transportation. 1406.
"The Railways' Part in the Development of Hampton Roads." Charles S. Churchill, 2006.

Frequency
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. 1699. Discussion: C. E. Grunsky, 2089.
"Theoretical Frequency Curves and Their Application to Engineering Problems." H. Alden Foster. 825. Discussion: Allen Hazen, 1292; Frederick H. Hapgood, Richard T. Dana, 1644; L. Standish Hall, R. D. Goodrich, 1751; John Tucker, Jr., H. Alden Foster, 1860.

Generators
See Electric Generators.

Geology
Engineering —— of the Catskill Water Supply. Discussion: William W. Brush, Thomas H. Wiggin, Joel D. Justin, 123; James F. Kemp, Thaddeus Merriman, Charles P. Berkey, James F. Sanborn, 568.
Surface —— of the lower Mississippi. 1177.

Harbors
Modern rail and water terminals with particular reference to the situation at Chicago, Illinois. 1552.
"The Railways' Part in the Development of Hampton Roads." Charles S. Churchill, 2006.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, John Klorer, John F. Coleman, Samuel McCain Young, John R. Freeman, 1133; Elliott J. Dent, 1722; M. J. Sanders, Joseph E. Ransdell, 2021. Discussion: B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, C. E. Grunsky, 1250; John R. Freeman, W. G. Price, J. A. Ockerson, Sidney F. Lewis, C. W. Chapin, C. McD. Townsend, 1624; W. M. Black, H. B. Muckleston, H. C. Ripley, H. F. Dunham, 1792; E. J. Dent, 1929; Lewis M. Haupt, C. E. Grunsky, William T. Lyle, 2045.

Highways
Development of —— in the United States, 1429.
"Proposed Loading for Highway Bridges." Harold D. Hussey. 1031. Discussion: J. A. L. Waddell, Otis E. Hovey, D. B. Steinman, R. de Charms, Samuel J. Ott, Lewis E. Moore, Glenn B. Woodruff, 1901.

Hydraulic Laboratories
The need of ——. 1185, 1250 *et seq.*, 1929.

Hydraulic Tests
"Comparative Tests on Experimental Draft-Tubes." 1813.
"The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models." Ford Kurtz. 1953.

Hydraulics
See Water, Flow of; Water Power; Turbines (Water).

Impact
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.

Instruments
—— used in experimental tests on draft-tubes. 1820.
—— used in impact studies on Skunk River Bridge, Ames, Iowa. 458 *et seq.*
—— used in measuring stresses in railroad track. 307 *et seq.*

Iron
See Cast Iron.

Irrigation
"A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California." S. L. Rothery. 671. Discussion: Paul M. Entenmann, 1285.

Jetties
Economic location of ——. 1141.

Laboratories
See Hydraulic Laboratories.

Land
Railroad terminal —— values. 1426.

Law
Operation of the Federal Water Power Act. 259.

Levees
"A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California." S. L. Rothery. 671. Discussion: Paul M. Entenmann, 1285.
—— on the Mississippi River. 1149, 1627 *et seq.*
Work of the Corps of Engineers on the lower Mississippi. 1134.

Loads
See Wheel Loads.

Locks
"Some Notes on the Location and Construction of —— and Movable Dams on the Ohio River, with Particular Reference to Ohio River Dam No. 18." Discussion: William M. Hall, 771.

Locomotives
"Locomotive Loadings for Railway Bridges." Discussion: Charles Evan Fowler, C. F. Loweth, Ralph Modjeski, C. A. P. Turner, 131; D. B. Steinman, 952.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Maps
See Aeronautic Maps.

Measuring Instruments
See Instruments.

Memoirs
Allen, Ralph Benjamin. 819.
Armstrong, William Coulson. 2091.
Arnoll, Kingsley Parkhurst. 2103.
Axtell, Decatur. 805.
Baldwin, Archibald Stuart. 589.
Bensel, John Anderson. 583.
Bolton, Channing Moore. 806.
Borden, Henry Percy. 279.
Britton, John Alexander. 1677.
Bryant, Byron Harkness. 594.
Carter, Shirley. 809.
Chamberlin, Chester Harvey. 1662.
Coane, Henry Edward. 1806.
Comber, William George. 1662.
Compton, Alfred George. 1680.
Cowles, Walter Linsley. 595.
Cresson, Benjamin Franklin. 1664.
de Varona, Ignacio Marea. 141.
Drake, William Abial. 1330.
Edes, William Cushing. 280.
Flaherty, Edward Thomas. 2093.
Foster, Wilbur Fisk. 143.
Gemmell, Robert Campbell. 150.
Gillespie, Richard Henwood. 810.
Grantham, Herbert Thomas. 811.
Griggs, Julian. 1667.
Hamilton, Frank Henry. 1332.
Hardwicke, Alan Hyde Gardner. 1357.
Hardy, George Richardson. 1940.
Harris, Archie Lee. 285.
Harrisson, Gerardus. 1351.
Healey, Charles Frank. 1333.
Hirsch, Herman David. 160.
Hodgdon, Frank Wellington. 1336.
Holland, Arthur Francis. 288.
Howell, William Augustus. 1026.
Hunt, Robert Woolston. 1668.
Jacob, Clarence Cecil. 1353.
Jaques, William Henry. 2094.
Kaufman, Gustave. 1341.
Kenyon, William John Charles. 1358.
Knap, Edgar Day. 1356.
Knapp, Louis Henry. 1941.
Krigbaum, Lowell Gaynor. 2104.
Lafler, William Arthur. 596.
Lantz, Clarence Ivan. 2106.
Lau, Percy Mapes. 598.
Leavitt, Erasmus Darwin. 2096.
Macnicol, John Alexander. 1341.
McNab, William. 1342.
Manley, Henry. 2097.
Marden, Henry Herman, Jr. 2099.
Mason, Sampson Douglas. 1343.
May, William Andrew. 1671.
Moore, Robert. 139.
Moulton, Mace. 811.
Newberry, Spencer Baird. 289.
Newman, Robert Morris. 813.
Parker, Maurice Stiles. 152.
Parker, William. 1943.
Pearson, Fred Stark. 2100.
Plympton, George Washington. 1672.
Prindle, Franklin Cogswell. 1944.
Rapp, Frank Arthur. 1673.
Reeves, David. 1346.
Rollo, Ralph Alexander. 1950.
Ross, Charles Wilson. 2107.
Sabin, Alpheus Timothy. 1348.
Scheuermann, Hugo Julius. 160.
Schwendener, Karl De Witt. 1027.
Seltzer, Harry Kent. 814.
Shirreffs, Reuben. 1946.
Shoemaker, John Earl. 820.
Sleeper, George Edward. 1948.
Smith, Hilmar Frederick. 820.
Sorzano, Julio Federico. 1675.
Stanton, Robert Brewster. 1807.
Tait, Clarence Everett. 1809.
Tait, William Stuart. 1349.
Taylor, Edward Ballinger. 817.
Tinkham, Samuel Everett. 153.
Uhler, William David. 282.
Wagoner, Luther. 155.
Washburn, Frank Sherman. 157.
Watson, Merrill. 1359.
Watson, William. 1949.
Watson, Winslow Barnes. 290.
Weber, George Adam. 1682.
Whistler, Thomas Delano. 159.
Wright, Charles Albert Loring. 597.
Yates, Preston King. 2102.
Young, Henry Amerman. 1676.
Zahniser, George Brown. 1350.

Metallurgy
Study of steels for engineering structures. 524.

Metals
Study of steels for engineering structures. 524.

Models
"Experiments with —— of the Gilboa Dam and Spillway." Discussion: B. F. Groat, Robert Fletcher, I. Gutmann, Robert F. Ewald, Thaddeus Merriman, 240; R. W. Gausmann, C. M. Madden, 566.

Mortars
"The Disintegration of Cement in Sea Water." William G. Atwood and A. A. Johnson. 1038. Discussion: Nathan C. Johnson, W. M. Black, Alfred D. Flinn, Benjamin A. Howes, Thomas H. Wiggin, Robert Ridgway, Richard H. Gaines, J. R. Slattery, Kenneth Allen, John Charles Riedel, Charles Rufus Harte, Chandler Davis, Alfred H. White, 1647; B. Jeanneret, A. H. Rhett, I. Hiroi, Theodore Belzner, 1762; E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, Jasper O. Draffin, 1914; Charles A. Newhall, Eduardo de Castro, R. Feret, 2060.

Movable Dams
See Dams.

National Research Council
"Engineering Foundation: Division of Engineering, of National Research Council: Their Origin, Work, Plans, Needs." Alfred D. Flinn. 518.

Parks
"—— and Parkways." 702.

Penstocks
"Elastic Stresses in the Rock Surrounding Pressure Tunnels." Charles P. Dunn. 661. Discussion: B. F. Jakobsen, Jacob Feld, William Cain, 1264.

Photography, Aerial
Aerial photography and map making with special reference to water power surveys, 117, 119, 120, 984, 987.

Pipe
Danger from pipes or conduits passing through earth dams. 900.

Ports
See Harbors.

Power
See Electric Power: Water Power.

Power Transmission
High voltage power transmission. 579.

Pressures
See Earth Pressures.

Probabilities, Theory of
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. 1699. Discussion: C. E. Grunsky, 2089.
"Theoretical Frequency Curves and Their Application to Engineering Problems." H. Alden Foster. 825. Discussion: Allen Hazen, 1292; Frederick H. Hapgood, Richard T. Dana, 1644; L. Standish Hall, R. D. Goodrich, 1751; John Tucker, Jr., H. Alden Foster, 1860.

Radio Communication
Radio and research. 548.

Railroad Terminals
"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. Ford, J. D'Esposito, Rufus W. Putnam, William L. R. Haines. 1405. Discussion: Clement C. Williams, 1799; B. F. Jakobsen, William T. Lyle, 1936.
Terminal land values. 1426.

Railroads
"Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada." J. P. Newell. 3. Discussion: J. G. Sullivan, 991.
"Final Report on Specifications for Design and Construction of Steel Railway Bridge Superstructure": Submitted by the Special Committee on Specifications for Bridge Design and Construction. 53.
"Locomotive Loadings for Railway Bridges." Discussion: Charles Evan Fowler, C. F. Loweth, Ralph Modjeski, C. A. P. Turner, 131; D. B. Steinman, 952.
"Progress Report of the Special Committee on Electrification of Steam Railroads." 488.
"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Fellheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. Ford, J. D'Esposito, Rufus W. Putnam, William L. R. Haines, 1405. Discussion: Clement C. Williams, 1799; B. F. Jakobsen, William T. Lyle, 1936.
"The Railways' Part in the Development of Hampton Roads." Charles S. Churchill, 2006.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Rails
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Rainfall
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. 1699. Discussion: C. E. Grunsky, 2089.

Reinforced Concrete
"—— Columns." John Tucker, Jr. 165. Discussion: Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, 787; H. V. Spurr, Austin H. Reeves, 999.

Reports of Committees
"Final Report on Specifications for Design and Construction of Steel Railway Bridge Superstructure." Submitted by the Special Committee on Specifications for Bridge Design and Construction. 53.
"Progress Report of the Special Committee on Electrification of Steam Railroads." 488.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations," etc., 1732.
"Tentative Specifications for Steel Highway Bridge Superstructure": Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Research
"Engineering ——": Technical Papers Presented at the Annual Meeting, January 19, 1923. Arthur N. Talbot, Alfred D. Flinn, George K. Burgess, Otto B. Blackwell, 511. Discussion: Clemens Herschel, William G. Atwood, H. H. Rousseau, W. C. Cushing, Charles Rufus Harte, F. E. Schmitt, Henry Goldmark, 746; W. A. Slater, 998.

Reservoirs
Borings at Ashokan Reservoir of the Catskill Water Supply System. 1744.
Effect on wave height of wind velocity and fetch in ——. 910 *et seq.*
"Tentative Plan for the Construction of a 780-Foot Rock-Fill Dam on the Colorado River at Lees Ferry, Arizona." Discussion: E. C. La Rue, 942.

Retaining Walls
"Lateral Earth Pressure: The Accurate Experimental Determination of the Lateral Earth Pressure, Together with a Résumé of Previous Experiments." Jacob Feld. 603. Discussion: J. C. Meem, E. G. Haines, George Paaswell, D. B. Steinman, M. J. McPike, Edward Godfrey, G. M. Braune, Thomas H. Wiggin, 1007.

Rivers
"A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California." S. L. Rothery. 671. Discussion: Paul M. Entenmann, 1285.
Aerial photography and map making with special reference to water power surveys, 117, 119, 120, 984, 987.
Maximum recorded floods in American ——. 1896.
"Some Notes on the Location and Construction of Locks and Movable Dams on the Ohio River, with Particular Reference to Ohio River Dam No. 18." Discussion: William M. Hall, 771.
"Tentative Plan for the Construction of a 780-Foot Rock-Fill Dam on the Colorado River at Lees Ferry, Arizona." Discussion: E. C. La Rue, 942.
The Colorado River development. 113, 114.
The Columbia River development. 115.
"The River and Harbor Problems of the Lower Mississippi": A Symposium. Lansing H. Beach, Henry Clay Ripley, A. L. Dabney, L. R. Parmelee, John Klorer, John F. Coleman, Samuel McCain Young, John R. Freeman, 1133; Elliott J. Dent, 1722; M. J. Sanders, Joseph E. Ransdell, 2021. Discussion: B. F. Groat, Hardy Cross, L. W. Wallace, L. K. Sherman, Morris Knowles, Gardner S. Williams, A. W. Newton, John Millis, C. E. Grunsky, 1250; John R. Freeman, W. G. Price, J. A. Ockerson, Sidney F. Lewis, C. W. Chapin, C. McD. Townsend, 1624; W. M. Black, H. B. Muckleston, H. C. Ripley, H. F. Dunham, 1792; E. J. Dent, 1929; Lewis M. Haupt, C. E. Grunsky, William T. Lyle, 2045.

Run-Off
"Periodic Fluctuations of Rainfall in Hawaii." Joel B. Cox. 1699. Discussion: C. E. Grunsky, 2089.
"Theoretical Frequency Curves and Their Application to Engineering Problems." H. Alden Foster. 825. Discussion: Allen Hazen, 1292; Frederick H. Hapgood, Richard T. Dana, 1644; L. Standish Hall, R. D. Goodrich, 1751; John Tucker, Jr., H. Alden Foster, 1860.

Sand
Tests with —— for earth pressure. 617.

Shore Protection
"Ocean Beach Esplanade, San Francisco, Calif." M. M. O'Shaughnessy. 1846.
See also Jetties.

Societies
See Engineering Societies.

Soils
"Lateral Earth Pressure: The Accurate Experimental Determination of the Lateral Earth Pressure, Together with a Résumé of Previous Experiments." Jacob Feld. 603. Discussion: J. C. Meem, E. G. Haines, George Paaswell, D. B. Steinman, M. J. McPike, Edward Godfrey, G. M. Braune, Thomas H. Wiggin, 1007.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of —— for Foundations," etc. 1732.

Spillway
"Experiments with Models of the Gilboa Dam and ——". Discussion: B. F. Groat, Robert Fletcher, I. Gutmann, Robert F. Ewald, Thaddeus Merriman, 240; R. W. Gausmann, C. M. Madden, 566.
—— into Lake Borgne, La. 1164.
"The Hydraulic Design of the Shaft —— for the Davis Bridge Dam and Hydraulic Tests on Working Models." Ford Kurtz. 1953.
Use of —— as a means of reducing flood heights. 1637.

Steam Turbines
See Turbines (Steam).

Steel
Study of steels for engineering structures. 524.
"Tentative Specifications for Steel Highway Bridge Superstructure": Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.

Strain
"Reinforced Concrete Columns." John Tucker, Jr. 165. Discussion: Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, 787; H. V. Spurr, Austin H. Reeves, 999.

Streets
Street development in relation to railroad terminals. 1519.

Strength of Materials
"Reinforced Concrete Columns." John Tucker, Jr. 165. Discussion: Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, 787; H. V. Spurr, Austin H. Reeves, 999.

Stress
"Elastic Stresses in the Rock Surrounding Pressure Tunnels." Charles P. Dunn. 661. Discussion: B. F. Jakobsen, Jacob Feld, William Cain, 1264.
"Improved Type of Multiple-Arch Dams." Fred A. Noetzli. 1065. Discussion: B. F. Jakobsen, Victor H. Cochrane, William Cain, 1767; H. deB. Parsons, Herman Schorer, 1924; H. J. F. Gourley, Alfred D. Flinn, 2067.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.
"Reinforced Concrete Columns." John Tucker, Jr. 165. Discussion: Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, 787; H. V. Spurr, Austin H. Reeves, 999.
"Stresses in Multiple-Arch Dams." B. F. Jakobsen. 1093. Discussion: William Cain, 1785; Lars R. Jorgensen, 1927; Fred A. Noetzli, 2069.
"Tentative Specifications for Steel Highway Bridge Superstructure": Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Structural Steel
See Steel.

Surveying, Aerial
Aerial photography and map making, with special reference to water power surveys. 117, 119, 120, 984, 987.

Terminals
See Harbors, Railroad Terminals.

Track
"Third Progress Report of the Special Committee to Report on Stresses in Railroad ——." 295. Discussion: S. Timoshenko, 1279.

Traffic
"Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada." J. P. Newell. 3. Discussion: J. G. Sullivan, 991.
Street development in relation to railroad terminals. 1519.
—— in relation to railroad terminals. 1459.

Transportation
"Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada." J. P. Newell. 3. Discussion: J. G. Sullivan, 991.
"Railroad Transportation and Railroad Terminals": A Symposium. John W. Kendrick, J. G. Sullivan, J. Rowland Bibbins, Edwin F. Wendt, Charles A. Morse, John S. Worley, Alfred Feilheimer, Charles E. Smith, Jacob L. Crane, Jr., D. J. Brumley, Frederick E. Morrow, Robert H. Ford, J. D'Esposito, Rufus W. Putnam, William L. R. Haines. 1405. Discussion: Clement C. Williams, 1799; B. F. Jakobsen, William T. Lyle, 1936.
The —— Act. 1412, 1432, 1440, 1446, 1448.
—— on Mississippi River. 2021.

Tunnels
"Elastic Stresses in the Rock Surrounding Pressure ——." Charles P. Dunn. 661. Discussion: B. F. Jakobsen, Jacob Feld, William Cain, 1264.
Pressure —— of the Catskill Aqueduct, 125.

Turbines (Steam)
"The Design of Structural Supports for Turbo-Generators." Edward H. Cameron. 37. Discussion: W. E. Belcher, Austin H. Reeves, F. A. Ericson, James H. Richardson, F. M. Beer, 274; George A. Orrok, 581; R. von Fabrice, 782; D. A. Allee, Edward H. Cameron, 993.

Turbines (Water)
"Comparative Tests on Experimental Draft-Tubes." 1813.

Turbo-Generators
See Electric Generators.

Valuation
Federal —— of railroads. 1432.
Railroad terminal land values. 1426.

Water, Flow of, in Pipes
"The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models." Ford Kurtz, 1953.

Water, Flow of, Over Dams and Weirs
"The Hydraulic Design of the Shaft-Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models." Ford Kurtz, 1953.

Water Power
"Comparative Tests on Experimental Draft-Tubes." 1813.
Operation of the Federal —— Act. 259.
"The —— Problem": A Symposium. Frederick Hall Fowler. 917. Discussion: J. B. Lippincott, C. S. Jarvis, D. C. Henny, F. H. Tibbetts, Louis C. Hill, C. H. Birdseye, Edward H. Sargent, 113; William T. Lyle, Howard R. Farnsworth, J. W. Swaren, 258; J. P. Jollyman, 579; F. P. Williams, 780; G. B. Herington, Gerard H. Matthes, 984.
—— development in New York State. 122, 258, 780.
—— potentialities of the Pacific Coast. 917.

Water, Underground
Movement of —— and its relation to the design of earth dams. 867.

Water-Works
"Engineering Geology of the Catskill Water Supply." Discussion: William W. Brush, Thomas H. Wiggin, Joel D. Justin, 123; James F. Kemp, Thaddeus Merriman, Charles P. Berkey, James F. Sanborn, 568.

Waterways
Development of —— in the United States. 1429.
Modern rail and water terminals with particular reference to the situation at Chicago, Illinois. 1552.
See also Rivers.

Waves
Effect on wave height of wind velocity and fetch in reservoirs. 910 *et seq.*

Wharves
—— at the port of New Orleans, La. 1168, 1182.
See also Docks.

Wheel Loads
Locomotive Loadings for Railway Bridges. Discussion: Charles Evan Fowler, C. F. Loweth, Ralph Modjeski, C. A. P. Turner, 131; D. B. Steinman, 952.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.
"Proposed Loading for Highway Bridges." Harold D. Hussey. 1031. Discussion: J. A. L. Waddell, Otis E. Hovey, D. B. Steinman, R. de Charms, Samuel J. Ott, Lewis E. Moore, Glenn B. Woodruff, 1901.
"Tentative Specifications for Steel Highway Bridge Superstructure": Submitted as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction. 1377. Discussion: Clarence W. Hudson, 1607; S. H. Leister, Theodore Belzner, L. H. Shoemaker, Adolf Eggenschwyler, 2072.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295. Discussion: S. Timoshenko, 1279.

Zoning
See City Planning.

AUTHOR INDEX

Alexander, Magnus W.
"The Objective in Engineering Education." 698.

Allee, D. A.
Design of structural supports for Turbo-generators. 993.

Allen, C. M.
"Comparative Tests on Experimental Draft-Tubes." 1813.

Allen, Kenneth
Disintegration of cement in sea water. 1655.

Allen, Ralph Benjamin
Memoir of. 819.

Ames, J. H.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.

Andrews, Carl B.
"Reactions for a Particular Type of Unsymmetrical Arch." 1685.

Armstrong, William Coulson
Memoir of. 2091.

Arnold, Bion J.
"Progress Report of the Special Committee on Electrification of Steam Railways." 488.

Arnoll, Kingsley Parkhurst
Memoir of. 2103.

Atwood, William G.
Engineering research. 747.
"The Disintegration of Cement in Sea Water." 1038.

Axtell, Decatur
Memoir of. 805.

Baldwin, Archibald Stuart
Memoir of. 589.

Beach, Lansing H.
"The Work of the Corps of Engineers on the Lower Mississippi." 1134.

Beer, F. M.
Design of structural supports for turbo-generators. 278.

Belcher, W. E.
Design of structural supports for turbo-generators. 274.

Belzner, Theodore
Disintegration of cement in sea water. 1766.
Tentative specifications for steel highway bridge superstructure. 2072.

Bensel, John Anderson
Memoir of. 583.

Berkey, Charles P.
"Engineering Geology of the Catskill Water Supply." 572.

Bibbins, J. Rowland
"Railroads—The Arteries of Commerce." 1417.

Birdseye, C. H.
Aerial photography as an aid in mapping. 120.

Black, S. B.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.

Black, W. M.
Disintegration of cement in sea water. 1648.
River and harbor problems of the lower Mississippi. 1792.

Blackwell, Otto B.
"Radio and Research." 548.

Bolton, Channing Moore
Memoir of. 806.

Borden, Henry Percy
Memoir of. 279.

Braune, G. M.
Determination of lateral earth pressure. 1021.

Bremner, G. H.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Britton, John Alexander
Memoir of. 1677.

Brumley, D. J.
"Chicago Terminal Improvements, Illinois Central Railroad." 1526.

Brunner, John
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Brush, William W.
Engineering geology, Catskill Water Supply. 123.

Bryant, Byron Harkness
Memoir of. 594.

Buck, H. R.
Engineering education. 744.

Buel, A. W.
Reinforced concrete colmuns. 802.

Burgess, George K.
"The Study of Steels for Engineering Structures." 524.

Burton, W. J.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Cain, William
Improved type of multiple-arch dams. 1777.
Stresses in multiple-arch dams. 1785.
Stresses in rock surrounding tunnels. 1275.

Cameron, Edward H.
"The Design of Structural Supports for Turbo-Generators." 37. Discussion, 994.

Caparn, Harold A.
City Planning. 769.

Carter, Shirley
Memoir of. 809.

Chamberlin, Chester Harvey
Memoir of. 1662.

Chapin, C. W.
River and harbor problems of the lower Mississippi. 1630.

Churchill, Charles S.
"The Railways' Part in the Development of Hampton Roads." 2006.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Coane, Henry Edward
Memoir of. 1806.

Cochrane, Victor H.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
Improved type of multiple-arch dams. 1768.
Reactions for a particular type of unsymmetrical arch. 2082.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.

Coleman, John F.
"The Port of New Orleans." 1167.

Comber, William George
Memoir of. 1662.

Compton, Alfred George
Memoir of. 1680.

Constable, Howard
Engineering education. 997.

Cowles, Walter Linsley
Memoir of. 595.

Cox, Joel B.
"Periodic Fluctuations of Rainfall in Hawaii." 1699.

Crane, Jacob L., Jr.
"Street Development in Relation to Railroad Terminals." 1519.

Cresson, Benjamin Franklin
Memoir of. 1664.

Cross, Hardy
River and harbor problems of the lower Mississippi. 1254.

Cummings, Robert A.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations," etc. 1732.

Cushing, W. C.
Engineering research. 750.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Cykler, Emil F.
Disintegration of cement in sea water. 1916.

Dabney, A. L.
"The Success of the Mississippi River Levees in the Flood of 1922." 1149.

Dana, Richard T.
Theoretical frequency curves and their application. 1646.

Davis, Chandler
Disintegration of cement in sea water. 1659.

Dawley, W. M.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

de Castro, Eduardo
Disintegration of cement in sea water. 2062.

de Charms, R.
Proposed loading for highway bridges. 1907.

Delano, Frederic A.
City planning. 760.

Dent, Elliott J.
River and harbor problems of the lower Mississippi. 1929.
"The Mouths of the Mississippi River." 1722.

D'Esposito, J.
"The Chicago Union Station Development." 1545.

de Varona, Ignacio Marea
Memoir of. 141.

Douglas, Walter J.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations," etc. 1732.

Draffin, Jasper O.
Disintegration of cement in sea water. 1922.

Drake, William Abial
Memoir of. 1330.

Dunham, H. F.
River and harbor problems of the lower Mississippi. 1796.

Dunn, Charles P.
"Elastic Stresses in the Rock Surrounding Pressure Tunnels." 661.

Edes, William Cushing
Memoir of. 280.

Eggenschwyler, Adolf.
Tentative specifications for steel highway bridge superstructure. 2072.

Eitzen, Arthur R.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.

Entenmann, Paul M.
Diversion on delta of the Colorado River. 1285.

Ericson, F. A.
Structural supports for turbo-generators. 276.

Ewald, Robert F.
Experiments with models of the Gilboa Dam and spillway. 244.

Farnsworth, Howard R.
The water power problem. 259.

Feld, Jacob
"Lateral Earth Pressure: The Accurate Experimental Determination of the Lateral Earth Pressure, Together with a Résumé of Previous Experiments." 603.
Reinforced concrete columns. 795.
Stresses in rock surrounding tunnels. 1268.

Fellheimer, Alfred
"Principles of Terminal Station Design." 1455.

Feret, R.
Disintegration of cement in sea water. 2066.

Field, John E.
Design of earth dams. 1874.

Finch, J. K.
Engineering education. 731.

Fisher, E. A.
City planning. 763.

Flaherty, Edward Thomas
Memoir of. 2093.

Fletcher, Robert
Engineering education. 729.
Experiments with models of the Gilboa Dam and spillway. 241.

Flinn, Alfred D.
Disintegration of cement in sea water. 1649.
"Engineering Foundation: Division of Engineering, of National Research Council: Their Origin, Work, Plans, Needs." 518.
Improved type of multiple-arch dams. 2067.

Ford Robert H.
"Chicago Terminal Improvements, Chicago, Rock Island and Pacific and New York Central Railways." 1539.

Foster, H. Alden
"Theoretical Frequency Curves and Their Application to Engineering Problems." 825. Discussion. 1862.

Foster, Wilbur Fisk
Memoir of. 143.

Fowler, Charles Evan
Locomotive loadings for railway bridges. 131.

Fowler, Frederick Hall
"Water Power Potentialities of the Pacific Coast." 917.
Freeman, John R.
River and harbor problems of the lower Mississippi. 1624.
"The Need of a National Hydraulic Laboratory for the Solution of River Problems." 1185.
Fuller, Almon H.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.
Gaines, Richard H.
Disintegration of cement in sea water. 1653.
Gausmann, R. W.
Experiments with models of the Gilboa Dam and spillway. 566.
Gemmell, Robert Campbell
Memoir of. 150.
Gibbs, George
"Progress Report of the Special Committee on Electrification of Steam Railways." 488.
Gillespie, Richard Henwood
Memoir of. 810.
Godfrey, Edward
Determination of lateral earth pressure. 1018.
Reinforced concrete columns. 787.
Goldmark, Henry
Engineering research. 752.
Goodrich, R. D.
Theoretical frequency curves and their application. 1759.
Gourley, H. J. F.
Design of earth dams. 1325.
Improved type of multiple-arch dams. 2067.
Grantham, Herbert Thomas
Memoir of. 811.
Greiner, J. E.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.
Griggs, Julian
Memoir of. 1667.
Groat, B. F.
Experiments with models of Gilboa Dam and spillway. 240.
River and harbor problems of the lower Mississippi. 1250.
Grün, Richard
Disintegration of cement in sea water. 1922.
Grunsky, C. E.
Periodic fluctuations of rainfall in Hawaii. 2089.
River and harbor problems of the lower Mississippi. 1263. 2056.
Gutmann, I.
Experiments with models of Gilboa Dam and spillway. 242.
Haines, E. G.
Design of earth dams. 1298.
Determination of lateral earth pressure. 1012.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing value of Soils for Foundations," etc. 1732.
Haines, William L. R.
"The Modern Freight Terminal of the Pennsylvania Railroad System in Chicago, Ill." 1585.
Hale, H. E.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.
Hall, L. Standish
Theoretical frequency curves and their application. 1751.
Hall, William M.
"Some Notes on the Location and Construction of Locks and Movable Dams on the Ohio River, with Particular Reference to Ohio River Dam No. 18." 771.
Hamilton, Frank Henry
Memoir of. 1332.
Hapgood, Frederic H.
Theoretical frequency curves and their application. 1644.
Harding, C. R.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.
Hardwicke, Alan Hyde Gardner
Memoir of. 1357.
Hardy, George Richardson
Memoir of. 1940.
Harrington, John L.
"Co-operation of National Engineering Societies in Engineering Education." 506.
Harris, Archie Lee
Memoir of. 285.
Harrisson, Gerardus
Memoir of. 1351.
Harte, Charles Rufus
Disintegration of cement in sea water. 1658.
Engineering research. 750.
Hatton, T. Chalkley
Engineering education. 737.
Haupt, Lewis Muhlenberg
River and harbor problems of the lower Mississippi. 2045.
Haydock, Charles
Design of earth dams. 1327.
Hazen, Allen
Design of earth dams. 1297.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing value of Soils for Foundations," etc. 1732.
Theoretical frequency curves and their application. 1292.
Healey, Charles Frank
Memoir of. 1333.
Hemstreet, George P.
City planning. 767.
Henny, D. C.
The water power problem. 115.
Hering, Rudolph
City planning. 765.
Herington, G. B.
Aerial photography as an aid in mapping. 984.
Herschel, Clemens
Engineering research. 746.
Hill, Louis C.
Aerial photography as an aid in mapping. 119.
Hiroi, I.
Disintegration of cement in sea water. 1764.
Hirsch, Herman David
Memoir of. 160.
Hirsch, John G.
Design of earth dams. 1744.
Hodgdon, Frank Wellington
Memoir of. 1336.
Holland, Arthur Francis
Memoir of. 288.

Holmes, J. Albert
Design of earth dams. 1317.

Hovey, Otis E.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
Proposed loading for highway bridges. 1903.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.

Howell, William Augustus
Memoir of. 1026.

Howes, Benjamin A.
Disintegration of cement in sea water. 1650.

Hudson, Clarence W.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377. Discussion. 1607.

Hughes, D. E.
Harbor improvements. 137.

Hunt, Robert Woolston
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.
Memoir of. 1668.

Hussey, Harold D.
"Proposed Loading for Highway Bridges." 1031.

Jackson, Dugald C.
Engineering education. 735.

Jacob, Clarence Cecil
Memoir of. 1353.

Jakobsen, B. F.
Improved type of multiple-arch dams. 1767.
Railroad transportation and railroad terminals. 1936.
"Stresses in Multiple-Arch Dams." 1093.
Stresses in rock surrounding tunnels. 1264.

Jaques, William Henry
Memoir of. 2094.

Jarvis, C. S.
Design of earth dams. 1619.
The water power problem. 114.

Jeanneret, B.
Disintegration of cement in sea water. 1762.

Jenkins, J. B.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Jewett, J. Y.
Disintegration of cement in sea water. 1920.

Johnson, A. A.
"The Disintegration of Cement in Sea Water." 1038.

Johnson, Nathan C.
Disintegration of cement in sea water. 1647.

Jollyman, J. P.
High-voltage power transmission. 579.

Jorgensen, Lars R.
Stresses in multiple-arch dams. 1927.

Justin, Joel D.
Engineering geology, Catskill Water Supply. 128.
"The Design of Earth Dams." 856. Discussion. 1883.

Kaufman, Gustave
Memoir of. 1341.

Kelley, E. F.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Progress Report of the Special Committee on Impact in Highway Bridges." 457.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.

Kemp, James F.
Engineering geology, Catskill Water Supply. 568.

Kendrick, John W.
"Some Phases of Present-Day Railroad Transportation." 1406.

Kenyon, William John Charles
Memoir of. 1358.

Ketchum, Milo S.
Engineering education. 740.
"Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Design and Construction of Steel Railway Bridge Superstructure." 53.
"Tentative Specifications for Steel Highway Bridge Superstructure." 1377.

Kittredge, George W.
"Progress Report of the Special Committee on Electrification of Steam Railways." 488.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Klorer, John
"The Flood Problem of the lower Mississippi River." 1159.

Knap, Edgar Day
Memoir of. 1356.

Knapp, Louis Henry
Memoir of. 1941.

Knowles, Morris
River and harbor problems of the lower Mississippi. 1259.
"Zoning—Its Progress and Application." 713.

Krigbaum, Lowell Gaynor
Memoir of. 2104.

Kurtz, Ford
"The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models." 1953.

La Bach, Paul M.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

Lafler, William Arthur
Memoir of. 596.

Lantz, Clarence Ivan
Memoir of. 2106.

Larsson, C. G. E.
"Third Progress Report of the Special Committee to Report on Stresses in Railroad Track." 295.

La Rue, E. C.
Tentative plan for the construction of a 780-foot rock-fill dam on the Colorado River at Lees Ferry, Arizona. 942.

Lau, Percy Mapes
Memoir of. 598.

Leavitt, Charles W.
City planning. 756.

Leavitt, Erasmus Darwin
Memoir of. 2096.

Leister, S. H.
Tentative specifications for steel highway bridge superstructure. 2072.

Lewis, Harold M.
City planning. 766.
Lewis, Nelson P.
City planning. 768.
"Regional Planning." 554.
Lewis, Sidney F.
River and harbor problems of the lower Mississippi. 1628.
Lippincott, J. B.
The water power problem. 113.
Loweth, Charles F.
Address at the Annual Convention in Chicago, Illinois, July 11, 1923. 1363.
Locomotive loadings for railway bridges. 132.
"Progress Report of the Special Committee on Electrification of Steam Railways." 488.
Lowrie, Charles N.
City planning. 761.
Lyle, William T.
City planning. 753.
Railroad transportation and railroad terminals. 1938.
River and harbor problems of the lower Mississippi. 2058.
The water power problem. 258.
Macnicol, John Alexander
Memoir of. 1341.
Macqueen, Philip O.
Comparison of concrete groined arches as an aid in their design. 575.
McCrosky, Theodore T.
Engineering education. 737.
McDaniel, A. B.
Engineering education. 744.
McNab, William
Memoir of. 1342.
McPike, M. J.
Determination of lateral earth pressure. 1018.
Madden, C. M.
Experiments with models of the Gilboa Dam and spillway. 566.
Manley, Henry
Memoir of. 2097.
Marden, Henry Herman Jr.
Memoir of. 2099.
Martin, E. S.
Reinforced concrete columns. 794.
Mason, Sampson Douglas
Memoir of. 1343.
Matthes, Gerard H.
Aerial photography as an aid in mapping. 987.
May, William Andrew
Memoir of. 1671.
Meem, J. C.
Determination of lateral earth pressure. 1007.
"Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations," etc. 1732.
Merriman, Thaddeus
Design of earth dams. 1744.
Engineering geology, Catskill Water Supply. 571.
Experiments with models of the Gilboa Dam and spillway. 249.
Millis, John
River and harbor problems of the lower Mississippi. 1262.
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Locomotive loadings for railway bridges. 134.
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City planning. 768.

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Locomotive loadings for railway bridges. 952.
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Engineering education. 733.

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The water power problem. 272.

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Memoir of. 1809.

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Memoir of. 1349.

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Memoir of. 817.

Tibbetts, F. H.
Aerial photography as an aid in mapping. 117.

Timoshenko, S.
Stresses in railroad track. 1279.

Tinkham, Samuel Everett
Memoir of. 153.

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Theoretical frequency curves and their application. 1860.

Turneaure, F. E.
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Locomotive loadings for railway bridges. 135.

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Memoir of. 282.

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Memoir of. 155.

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City planning. 755.

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Memoir of. 157.

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Memoir of. 1359.

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Memoir of. 290.

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Memoir of. 159.

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Wickenden, W. E.
Engineering education. 741.

Wiggin, Thomas H.
Concrete groined arches. 251.
Design of earth dams. 1313.
Determination of lateral earth pressure. 1023.
Disintegration of cement in sea water. 1652.
Engineering geology, Catskill Water Supply. 124.

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The water power problem. 780.

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Memoir of. 2102.

Young, Henry Amerman
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Young, Samuel McCain
"The Port Development Problem of the Lower Mississippi River." 1177.

Zahniser, George Brown
Memoir of. 1350.

THE PAPERS OF THE SOCIETY

In the first Constitution of 1852, and again in the new Constitution of 1921, the advancement of Engineering and of Architecture is defined as one of the objects of this Society. To assist in fulfilling this object, the Society began in 1872 to publish technical papers and discussions in *Proceedings* and *Transactions*. During the fifty years from then until now 1538 papers have been incorporated in the literature of the Profession.

In order to maintain high standards of excellence in papers accepted for publication, the Standing Committee on Technical Activities and Publications has been given supervision of this phase of the Society's activities. The Committee is authorized to make general rules for the presentation and preparation of papers, to obtain the opinion of experts as to their permanent technical value, and to publish such papers of merit as the limited funds of the Society permit. All matters connected with the consideration of papers are held strictly confidential.

For guidance in the preparation and presentation of manuscripts, the following suggestions are given:

1.—All manuscripts should be typewritten and double spaced.

2.—At least the original and one copy of the paper should be submitted.

3.—Letter paper of ordinary thickness rather than thin manifold paper should be used.

4.—Drawings or photographs that are intended for use as illustrations should not be inserted in the manuscript, but rather kept separate for the convenience of the engravers. All illustrations submitted should be numbered and referred to in the text by the proper numbers. Every attempt should be made to avoid sending with papers superfluous illustrations which do not add information of real value, since the cost of the illustrations is relatively high.

5.—Tables should be numbered and typewritten in the proper places in the body of the manuscript. Blue-printed tables should not be submitted.

6.—The paper should be prefaced by a brief synopsis giving the objects of the paper, a brief digest of its contents, and a concise statement of its conclusions.

7.—The Committee has ruled that those papers containing matter readily found elsewhere, those specially advocating personal interests, those carelessly prepared or controverting established facts, and those purely speculative or foreign to the purposes of the Society, shall be rejected.

8.—General discussions on papers are closed three months after publication. An additional month is granted for authors' closures.

9.—For further information, the rules for the preparation and presentation of papers on pages 44 and 45 of the 1923 Year Book should be consulted.

The Committee on Technical Activities and Publications cordially invites every member of the Society to co-operate in its work. Suggestions in regard to engineering projects and investigations which would provide proper subjects for papers of the requisite high order of excellence will be welcomed. Members and non-members alike are urged to submit papers for the consideration of the Committee, to the end that the horizon of engineering knowledge may steadily be widened and extended.

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

January 14, 1924:

A Quarterly Meeting will be held at Society Headquarters, New York, N. Y.

MONTHLY MEETINGS

December 5, 1923:

8:00 P. M.—A Joint Meeting with the American Society of Mechanical Engineers and the American Institute of Electrical Engineers to discuss the subject of "Hydro-Electric Power", will be held in the Auditorium of Engineering Societies Building, 33 West 39th Street, New York, N. Y.

February 6, 1924:

8:00 P. M.—A regular business meeting of the Society will be held, the program for which will be announced later.

March 5, 1924:

8:00 P. M.—A regular business meeting of the Society will be held, and a paper by Ford Kurtz, M. Am. Soc. C. E., entitled "The Hydraulic Design of the Shaft Spillway of the Davis Bridge Dam, and Hydraulic Tests on Working Models", will be presented for discussion. This paper is printed in this number of *Proceedings*.

ANNUAL MEETING

January 16, 17, and 18, 1924:

Seventy-first Annual Meeting of the Society.

January 16, 1924:

9:00 A. M.—Social Hour.

10:00 A. M.—Annual Meeting, Conferring of Honorary Memberships, and Presentation of Medals and Prizes for Papers.

2:30 P. M.—Presentation and Discussion of Committee Reports.

7:30 P. M.—President's and Honorary Members' Reception and Dinner Dance.

January 17, 1924:

9:00 A. M.—Social Hour.

10:00 A. M. to 4:30 P. M.—Conference of Local Section Representatives and Meetings of Technical Divisions.

8:00 P. M.—Address and Smoker.

January 18, 1924:

All-Day Excursion.